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Technical Report
TR-2103-SHR

SEISMIC CRITERIA FOR CALIFORNIA MARINE OIL TERMINALS

VOLUME 3
DESIGN EXAMPLE

by

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May 2000

Sponsored by

Marine Facilities Division
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13. ABSTRACT (Maximum 200 words)
The Navy and the California State Lands Commission entered into a Cooperative Research and Development Agreement for the development of seismic design criteria for waterfront construction. This task has produced NFESC TR-2103-SHR "Seismic Criteria for California Marine Oil Terminals" by Ferritto et al.; Volumes 1 and 2 dated July 1999. The document develops and expands on work that was begun by the Naval Facilities Engineering Service Center to provide seismic design criteria for waterfront construction. The cited report presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals and includes in addition to the criteria, seven chapters and three appendices of technical supporting material. The development of the criteria recognized the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State, and the economics of operating a commercial facility in a competitive market This report is the third volume of the guide and presents a design example case study prepared by Professor M. J. N. Priestley, University of California, San Diego. The design example uses the criteria and performance limits in the evaluation of a typical wharf structure. The procedures used in this example are presented in Chapter 3 of Volume of the Guide.

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EXECUTIVE SUMMARY

The Navy and the California State Lands Commission entered into a Cooperative Research and Development Agreement for the development of seismic design criteria for waterfront construction. Both organizations face similar problems in the safe design of facilities and the need for a design guide. The California State Lands Commission (CSLC) has oversight of over sixty marine oil terminals, some of which are over 80 years old and built to unknown standards. Typically, they were built to resist minor earthquake intensity. New earthquake hazard information from recent events such as Loma Prieta (1989) and Northridge (1994) indicate that much higher intensities are possible. It is prudent that these facilities be evaluated and unsafe deficiencies corrected.

This task has produced NFESC TR2103-SHR "Seismic Criteria for California Marine Oil Terminals" by Ferritto et al.; Volumes 1 and 2 dated July 1999. The document develops and expands on work that was begun by the Naval Facilities Engineering Service Center to provide seismic design criteria for waterfront construction. The cited report presents criteria that are intended to define a minimum level of acceptable performance for marine oil terminals and includes in addition to the criteria, seven chapters and three appendices of technical supporting material. The development of the criteria recognized the need to protect the environment from oil spills, the need to provide for the transfer of required natural resources into the State and the economics of operating a commercial facility in a competitive market. The development of this guide has taken the approach of providing reasonable and prudent levels of design consistent with the state-of-the-art of engineering practice. The document is intended to be dynamic in nature; it is expected that it will be revised and updated by the experience gained through usage.

This report is the third volume of the guide and presents a design example case study prepared by Professor M. J. N. Priestley of the University of California, San Diego. The design example uses the criteria and performance limits in the evaluation of a typical wharf structure. The procedures used in this example are presented in Chapter 3 of Volume of the Guide.

SEISMIC ASSESSMENT OF TW0-SEGMENT MARGINAL WHARF

THE STRUCTURE:

The structure is illustrated in Fig.1. The reinforced concrete deck consists of two segments, each 110 ft wide by 400ft long, connected by a central shear key, and has a uniform depth of 33in. Five lines of prestressed piles (A through F) support the deck at 20 ft centers in the transverse direction. Support in the longitudinal direction is also at 20 ft centers along all except line F, the landward row, where the spacing is 10 ft. The free ground surface provides a minimum clearance of 2 ft (at the landward edge, and for 5 ft beyond line F towards line E), then slopes at 1:1.75 uniformly. Clear heights of the piles are indicated on Fig.1. A cut-off wall at the landward edge of the wharf is pinned to the bottom of the wharf deck, and allows the ground surface beyond the wharf to be level with the deck surface.

The prestressed piles are 24 in. diameter circular piles, prestressed with 16-0.6 in. strands, as shown in Fig. 2. Strand area is 0.215 sq.in, UTS is 270 ksi, and stress after transfer and losses is 216ksi. For this case study, two details of transverse spiral reinforcement are considered to illustrate aspects of ductility and strength capacity. The first, W20 (A_{sp} = 0.20 in^2) @ 2.5 in. pitch represents good confinement as expected from new design. The second, W11 (A_{sp} = 0.11 in^2) @ 6.0 in. pitch is representative of older, substandard design. Spirals are ASTM A82 steel, with a yield stress of 70 ksi, and cover to the spirals is 3.0 in. Concrete strength, including strength gain with age is assessed to be 7.0 ksi. Piles are founded on a firm gravel layer at a depth of 120 ft below deck level.

Connection between piles and deck is by 8#10 grade 60 rebar dowels, with an assessed yield strength of 66 ksi. Connection details for the two cases considered are shown in Fig. 3. In the case of the well confined pile, spirals are continued into the deck, and the dowels are terminated by straight bar extensions 3 in. below the top of the deck. In the case of the older piles, no spirals are provided in the deck, and the dowels are bent outwards 12 in. below the deck surface to provide anchorage. This is expected to provide joint shear problems.

GROUND CONDITIONS

The foundation consists of quarry run overlaid by 3 ft of rip-rap. Geotechnical advice has resulted in an assessment that maximum soil resistance is reached at a soil displacement of 0.5 in., regardless of depth, and that maximum soil strength increases linearly with depth according to the relationship

$$p_{ult} = 0.5z \text{ kip/in} \qquad (1)$$

Where z is the depth below ground surface in ft, and p_{ult} is the maximum soil resisting force on a 1.0 in. height of pile. Thus, at a depth of 20 ft, the maximum soil resistance is

$$p_{20} = 0.5\text{x}20 = 10 \text{ kip/in.}$$

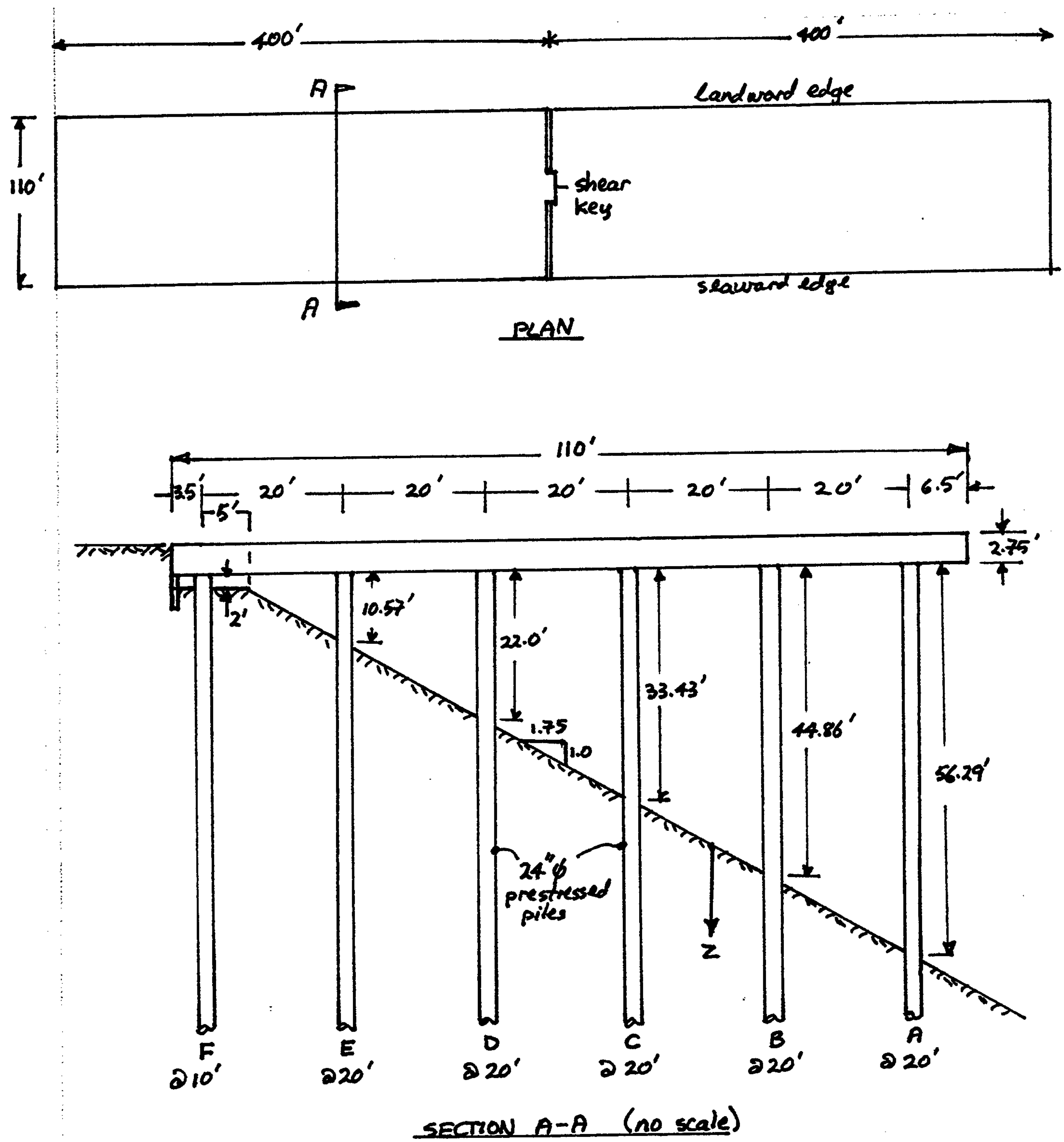
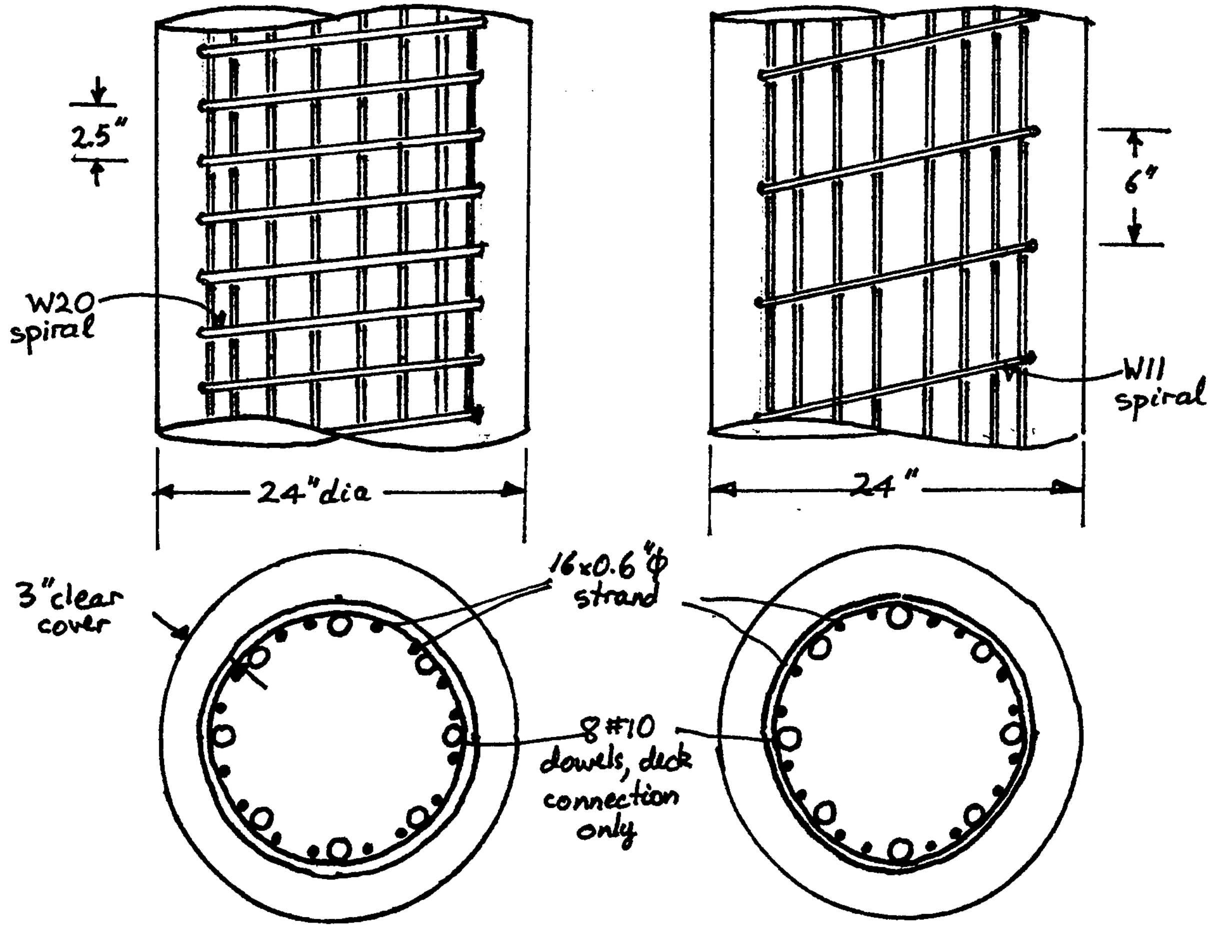


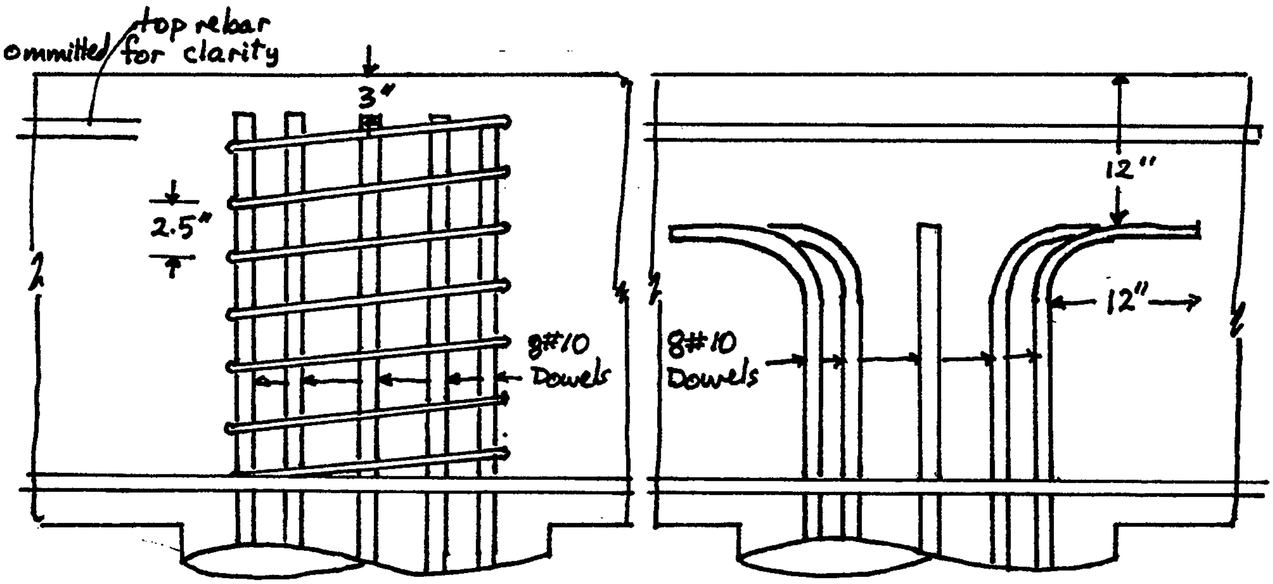
FIG 1. WHARF DIMENSIONS



(a) W20 @ 2.5"

(b) W11 @ 6"

FIG 2 PRESTRESSED PILE DETAILS



(a) W20 @ 2.5"

(b) No joint rebar; bent hooks

FIG 3 : CONNECTION DETAILS

If a soil spring represents a tributary pile height of 2 ft, the maximum spring force is thus

$$F_{20} = 10x24 = 240 \text{ kips},$$

And, since the peak resistance is reached at a displacement of 0.5 in., the elastic spring stiffness is

$$k_{20} = 240/0.5 = 480 \text{ kip/in.}$$

Soil spring characteristics are assessed to be elasto-plastic.

SEISMICITY

The structure is to be assessed for Level 2 (Damage control) response only, in accordance with the design spectrum of Fig. 4 which is tabulated in Table 1. Some tentative conclusions related to a level 1 earthquake whose intensity, for this case study, is assumed to be 40% that of the level 2 earthquake will also be made. The assessment will primarily be based on response spectrum analyses, but a time-history analysis using a real earthquake record scaled to have similar spectral response in the period range of interest will also be considered.

RESULTS

Detailed calculations are included in a Calculations Appendix. Representative listings of computer input and output are also included in a Computer Input/Output Appendix. This report contains only a summary of these detailed results, and describes decisions made in the analysis process.

Structural Weight:

For computation of the effective structural weight, one third of the pile weight for a length of pile from the deck soffit to 10ft (5xD) below the rip-rap surface is added to the deck weight. A tributary length of the wharf of 20 ft is considered, for which the weight, including self-weight, a uniform piping and equipment load of 35 psf, and tributary weight of the piles is found to be

$$W = 1022.3 \text{ kips/20 ft.}$$

The center of weight is 55.44 ft from the landward edge. (see pC1)

Seismic Axial Force in Piles

Estimates of the range of seismic axial forces in the piles are based on an initial estimate of the pile moment capacity of about 6000 kip.in. (see page C2). The procedure followed on pC2 is based on the following steps:

- Assume all piles have a pile top moment of 6000 kip.in, at the deck centerline

TABLE 1 475 YEAR DESIGN SPECTRUM (xg) (5% damping)

PERIOD	ACC. Xg
0	0.52
0.03	0.52
0.1	1.06
0.2	1.38
0.3	1.37
0.4	1.26
0.5	1.15
0.6	1.064
0.7	0.978
0.8	0.892
0.9	0.806
1	0.72
1.5	0.548
2	0.38
2.5	0.328
3	0.223
4	0.17

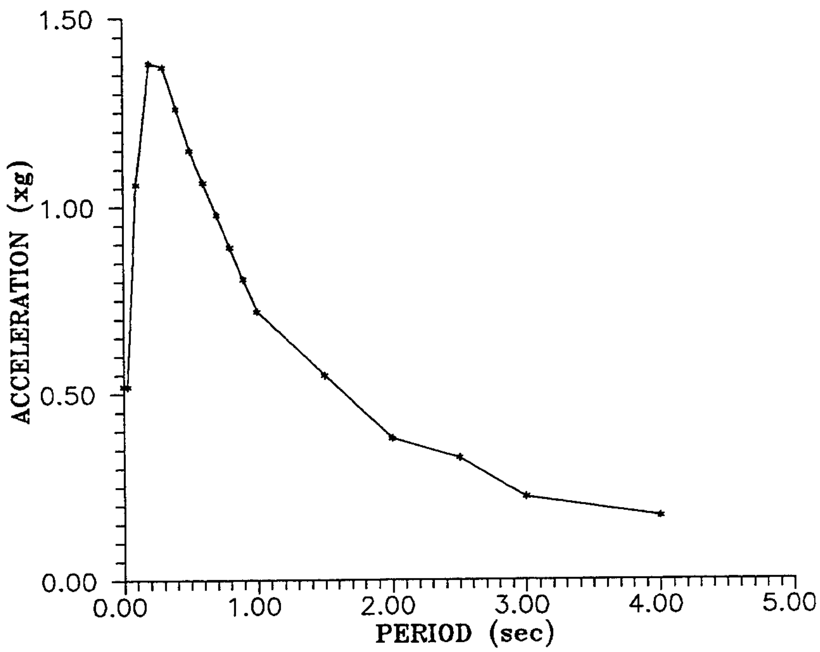


FIG.4 DESIGN ACCELERATION SPECTRUM
(5% damping)

- For equilibrium, deck moment at piles A and F =6000kip.in
- For interior piles (B to E), assume pile moment equally divided to each side on deck (i.e 3000 kip.in each side)
- At pile F, there are two piles each 20ft, so effective total deck moment there is 12000kip.in/20ft.
- Beam shear is found as the sum of beam moments at the two ends of a span divided by the span length (see eqns, pC2)
- For equilibrium, sum of beam shears on either side of a pile = seismic axial force. Note that for interior piles the beam shears effectively cancel, to result in only small axial force changes. At F, the beam shear V_{EF} is divided between the two F line piles in the 20ft width.

Pile Moment-Curvature calculations:

Calculations for these are included in pages C2 to C7. Data for the moment-curvature analyses and listings of the output are given in sub-appendix A of these calculations. Total axial force on the piles is estimated to vary between about 0 and 200 kips, and calculations are carried out for three levels of force: 0, 100, 200 kips, as a consequence. The programs used for moment-curvature analysis include the effects of enhancement to concrete compression strength and ultimate compression strain capacity resulting from confinement provided by the spirals, and differentiate between the unconfined cover concrete and the confined core. This is very important in this example, as the cover is a large portion (47% to center of the spiral) of the section area, and spalling of the cover at comparatively low strains dramatically influences the moment-curvature response. This is illustrated in the moment-curvature curves for the prestressed sections and the doweled connection detail which are shown in Fig. 5-8. As a consequence of cover spalling, ultimate moment capacities tend to be less than the peak values before cover spalling, particularly for the prestressed section when confined by the lighter W11 spiral.

The curves plotted in Figs 5-8 continue to the curvatures corresponding to the ultimate strains corresponding to the damage control (Level 2) limit state. Calculations for ultimate compression strains are included on pC3. Note that the ultimate curvatures are much lower for the section confined by the W11 spiral than when confined by the W20 spiral. Note also that the ultimate curvature is reduced as the axial force on the section increases. However, over the range of axial load considered, the influence of axial force on moment capacity and ultimate curvature is not great. A summary of limit state moments and curvatures is included on p C6 of the calculations. The moments and curvatures for the damage control limit state for the prestressed section with W20 spirals @ 2.5 in. correspond to the calculated ultimate strain capacity of 0.021. This exceeds the **design** level of 0.008 recommended for in-ground hinges (see p3-62 of resource document). However, for safety evaluation of an existing structure, the higher strain corresponding to unacceptable damage is probably more appropriate than the lower limit, which is intended to guard against long-term corrosion following a seismic event.

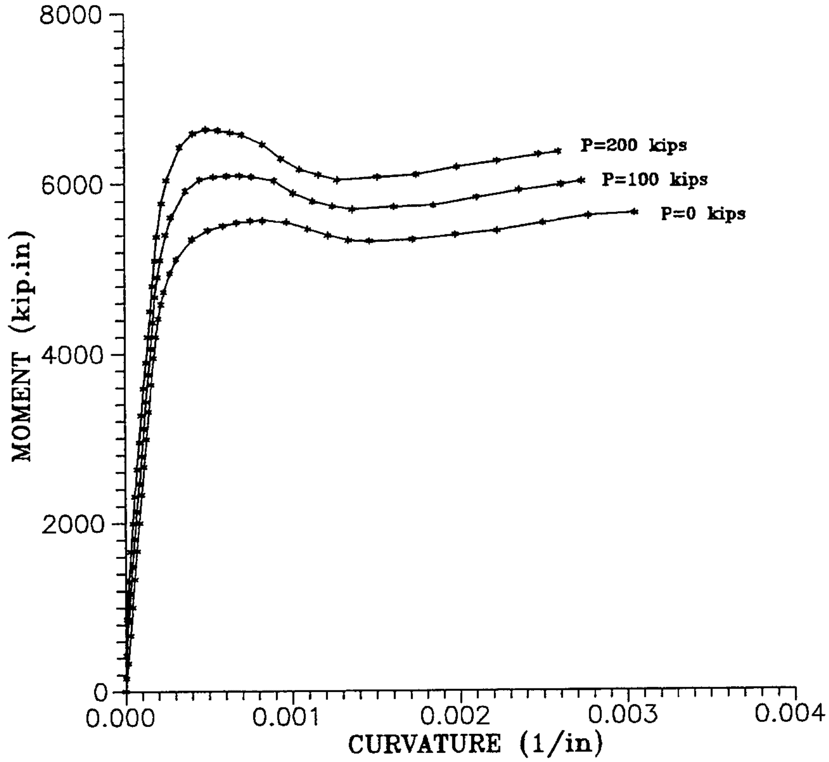


FIG 5. PILE TOP MOMENT-CURVATURE (W20/2.5in)

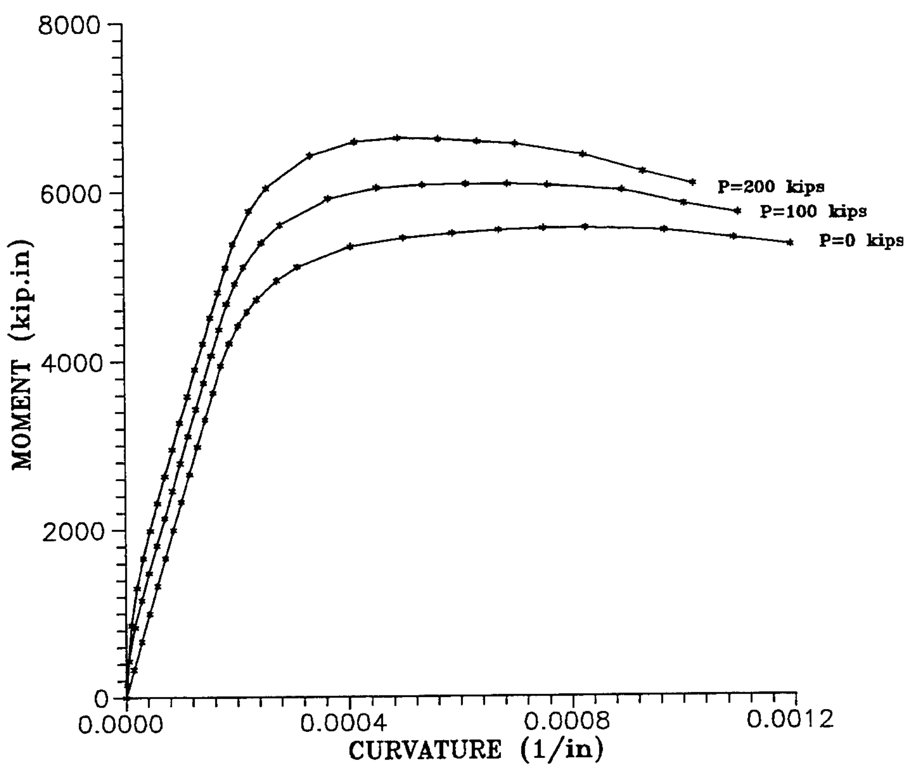


FIG 6. PILE TOP MOMENT-CURVATURE (W11/6in)

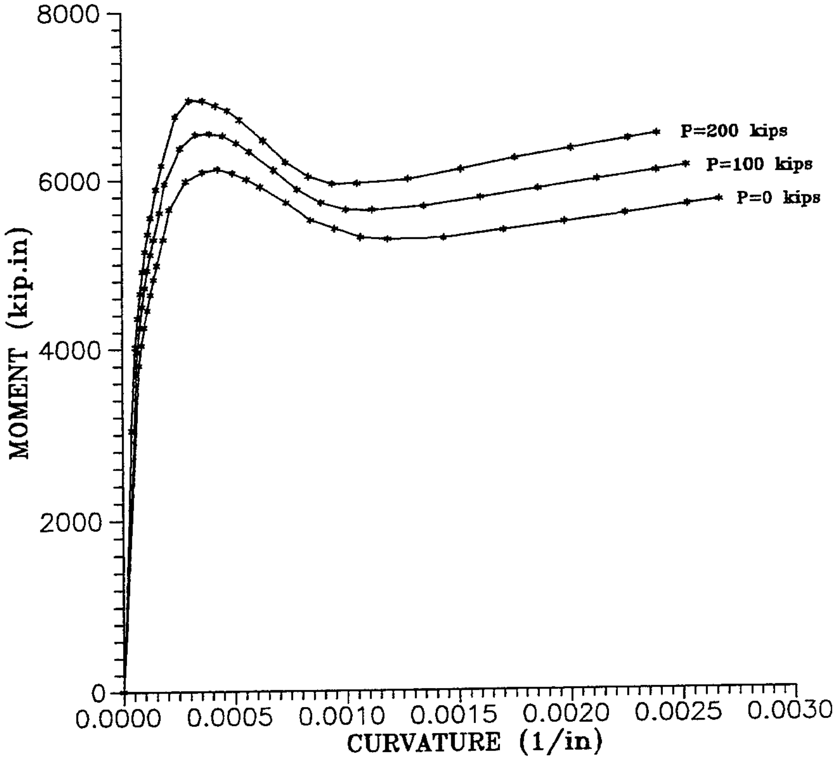


FIG 7. PRESTRESSED PILE MOMENT-CURVATURE (W20/2.5)

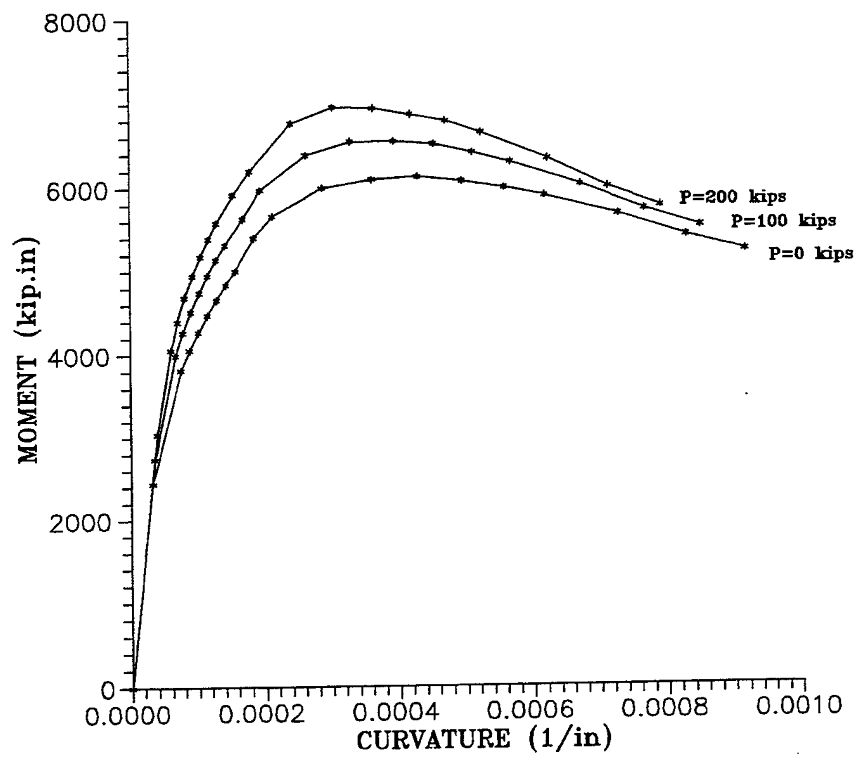


FIG 8. PRESTRESSED PILE MOMENT-CURVATURE (W11/6)

Pile Pushover Analyses:

In comparison with the piles, even the F-line piles, the 33 in thick deck is essentially rigid. This means that it is not necessary to model the deck flexibility in the pushover analyses. Section 4 of the calculations reports pushover analyses of the 6 pile heights (A to F) using two methods of analysis – inelastic Winkler foundation plus inelastic piles, and an equivalent depth to fixity analysis (pile F only). For the first case it is necessary to approximate the pile moment-curvature curves shown in Figs. 5 to 8 by equivalent simpler curves. The pile-top moment-curvature relationships for W20 spirals at 2.5 in centers can be adequately represented by elasto-plastic relationships (see p.C9). The comparison for the critical F-Line pile is shown in Fig. 9.

Because prestress delays cracking of the prestressed sections until the moment is close to 50% of ultimate capacity, the elastic portion of the moment-curvature section for prestressed sections is represented by a bi-linear relationship (pC10 to pC12). The inelastic portion is represented by a perfectly plastic curve. A comparison between the calculated moment-curvature relationship and an equivalent tri-linear (Muto) approximation is shown in Fig. 10. The agreement is very good in the elastic stage, but considerable differences exist in the plastic due to strength loss resulting from cover spalling, followed by strength increase from strain hardening. Note that the strength loss will probably be less than predicted by the moment-curvature analysis, since this does not include the influence of soil confinement on the cover concrete, which can be expected to delay, and reduce the influence of, cover spalling.

Spring-model for Inelastic Pushover Analysis: The pushover is carried out on a pile-by-pile basis, as suggested on pp3-54 and Fig.3-23 of the resource document. A similar model was used for all piles, with the basic difference being the clear height between the deck soffit and the ground surface. The basic model used is shown on pC13 of the calculations. Note that the top fixity of the pile is assumed to occur at a height of 12.6in above the soffit to model the additional flexibility caused by strain penetration.

The top elements of the pile are modeled by the deck-connection characteristics (figures on pC9), with lower elements modeled by the prestressed section characteristics (figures on pC12). For the F-Line pile members 16-19 have the connection characteristics, while for other piles, the top 4ft plus the strain penetration element have the connection characteristics. Note that the strain-penetration element is elastic- this forces the pile top plastic hinge to form at the soffit.

The soil springs have the bilinear characteristics defined in the section above on Ground Conditions. Calculations for the spring stiffnesses are included in pC14. The bottom 5 springs are given elastic characteristics, since the displacements at these depths will be much less than the yield displacement of 0.5 in.

Pushover analyses tend to become unstable when the structure forms a full inelastic mechanism, as occurs when hinges have formed at the deck soffit and at some depth in-ground. It can be difficult to determine the maximum strength and displacement capacity

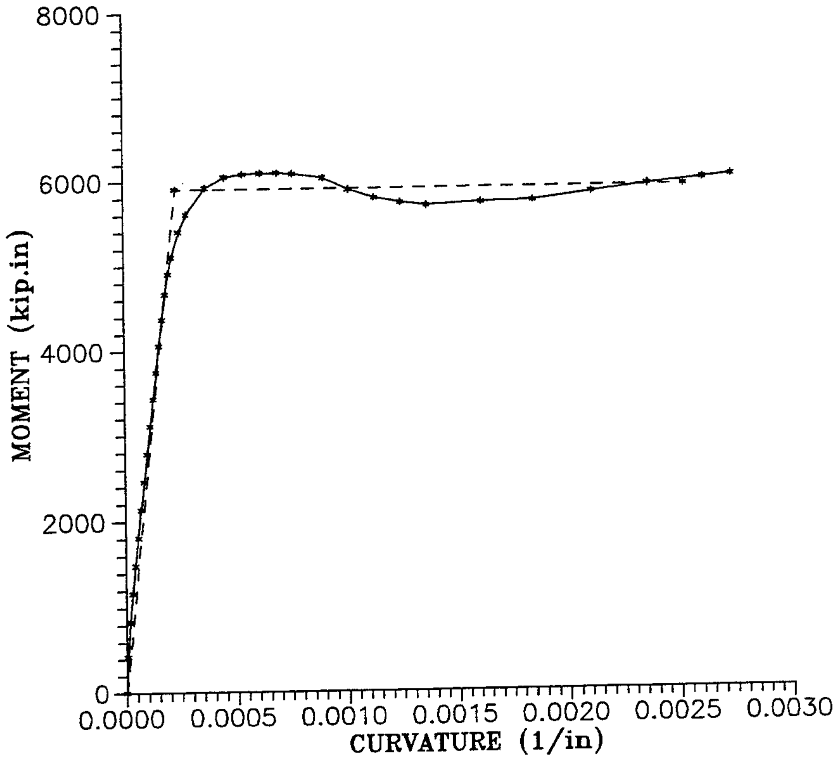


FIG. 9 PILE TOP MOM-CURV.+ ELASTO-PLASTIC APPROX
P = 100 kips

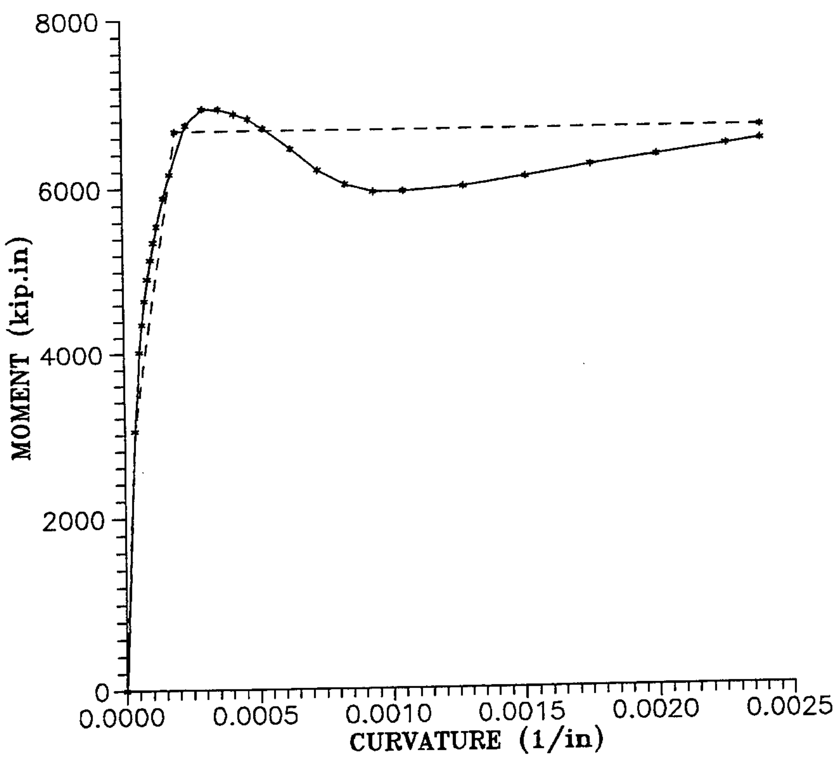


FIG. 10 PRESTRESSED PILE MOM-CURV. PLUS
MUTO (tri-linear) APPROX.(P=200kips)

with accuracy in these cases. To avoid this problem, a stiff spring (member 35, pC13)was placed in parallel with the pile at the pile top. This meant that the structure would always have positive stiffness, even when the pile had fully plastified. Note that this trick will also work if the pile has a strain-softening (negative second slope stiffness) characteristic after yield.

There are several inelastic pushover computer programs available. We have used an Inelastic time-history program "Ruaumoko" for all structural analyses, for convenience. It is possible to "fool" an inelastic time history analysis program to perform a pushover analysis by specifying a very slowly increasing acceleration ramp, which generates gradually increasing inertia forces on the structure. This has to be done slowly enough so that the higher modes of the structure are not excited. Ruaumoko has special facilities to enable pushover analyses, but it is emphasised that any inelastic time-history analysis code could be used if a special purpose pushover program is not available.

Input data for the pushover analysis for the F-Line pile is included in pp A-19a to A-19c. The data echo, which helps to explain the input, is listed in pp A-19d to A-19j. Listings for the force displacement response of each of the piles is also included in Appendix A, in pp A-20 to A-25 for piles A to F respectively. These data are also plotted in Fig. 11, and listed against a unified displacement base in Table 2 (located after Fig.11). Note that the data in Table 2 only are provided to a maximum displacement of 8.55in. This is calculated to be the ultimate displacement capacity of the critical F-line piles, based on the pile top hinge. Calculations for the displacement limits are given in ppC15 to C17. Note that these limits are only calculated for the F-line pile, since the larger displacement capacities if the other piles are of only academic interest. It will be noted that the displacement capacity of piles with W11 spirals at 6 in pitch, at 3.63in (pC16) is less than half that for the W20 @ 2.5in pitch.

Examination of Fig. 11 shows the very great differences in stiffness between the F line and other piles. It is also apparent from this figure that the yield displacements for the piles greatly increase as the clear height increases. This means that the final strengths of the piles, though still exhibiting high variations, are not as pronounced as variations in initial stiffness, which indicated that calculations based on initial stiffness are likely to be unreliable.

Fig. 12 shows moment profiles in the upper regions of the F-line pile at displacements close to first yield (0.94 in) and at maximum expected displacement (4.22 in). It is seen that the in-ground hinge forms at about 90 in (3.75 D) below ground surface. Elastic analyses would predict a lower height. The early yield of the weak upper soil springs means that the hinge is forced lower in the pile. It will be seen also that the moment profile close to the hinge varies only gradually, supporting the adopted plastic hinge length of 1.8D

For interest, Fig.13 plots moment profiles in the upper regions of the piles for Piles F, E and D, at displacements corresponding to full mechanism development. It will be noted that the in-ground hinge forms much higher in the longer piles. This is because the

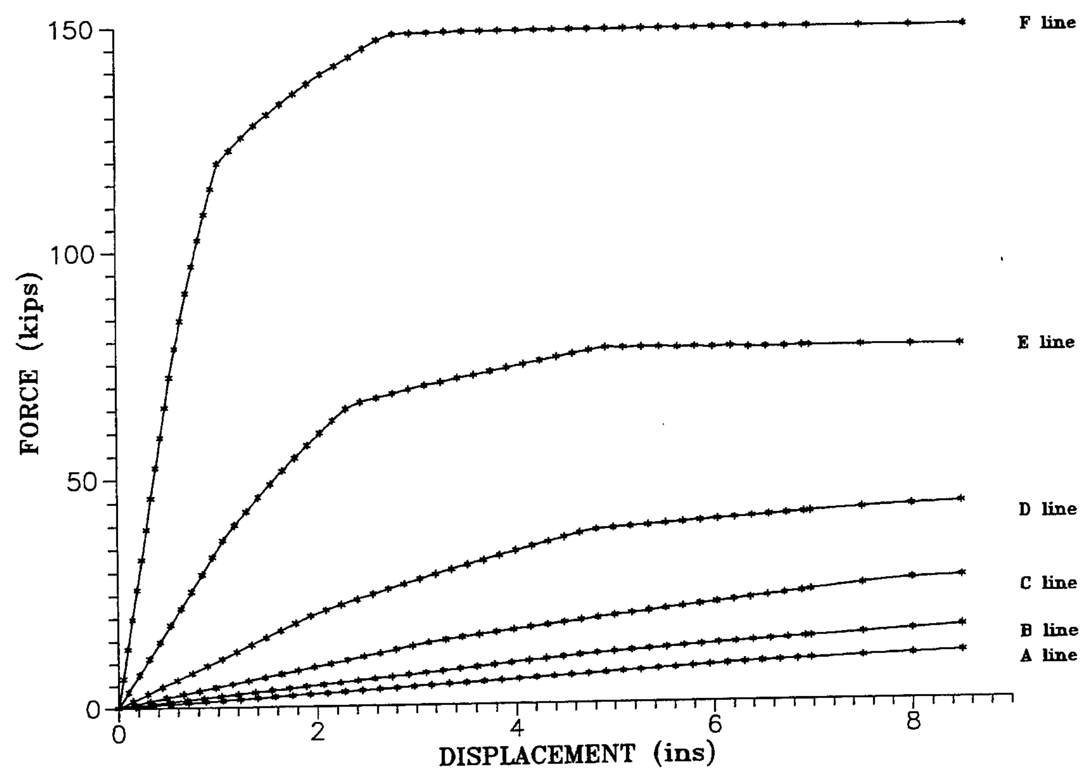


FIG. 11 PILE FORCE/DISPLACEMENT RESPONSE

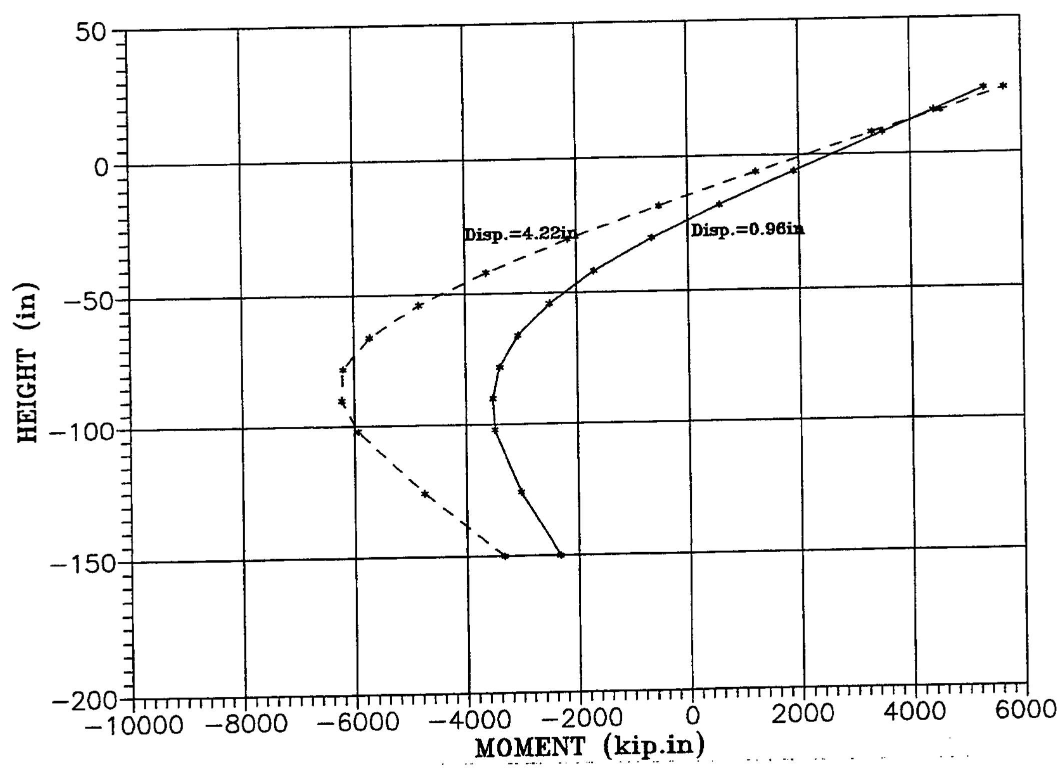


FIG. 12 F-LINE MOMENTS AT DIFFERENT DISPLACEMENTS

TABLE 2 PILE FORCE DISPLACEMENT RESPONSE

DISP	F	E	D	C	B	A	TOTAL	FED	CBA
0	0	0	0	0	0	0	0	0	0
0.4	53.93	13.88	4.15	1.78	0.93	0.55	129.15	125.89	3.26
0.6	79.71	20.82	6.22	2.67	1.4	0.82	191.35	186.46	4.89
0.8	100.19	27.76	8.3	3.56	1.86	1.1	242.96	236.44	6.52
1	117.38	34.69	10.37	4.45	2.33	1.37	287.97	279.82	8.15
1.2	123.63	40.53	12.44	5.35	2.79	1.65	310.02	300.23	9.79
1.4	128	45.45	14.52	6.24	3.26	1.92	327.39	315.97	11.42
1.6	131.56	50.04	16.59	7.13	3.72	2.19	342.79	329.75	13.04
1.8	134.94	54.54	18.67	8.02	4.19	2.47	357.77	343.09	14.68
2	138.14	58.75	20.74	8.91	4.65	2.74	372.07	355.77	16.3
2.2	140.97	62.91	22.16	9.8	5.12	3.02	384.95	367.01	17.94
2.4	143.61	65.8	23.61	10.69	5.58	3.29	396.19	376.63	19.56
2.6	146.29	67.09	24.97	11.58	6.05	3.57	405.84	384.64	21.2
2.8	148.15	68.16	26.31	12.47	6.52	3.84	413.6	390.77	22.83
3	148.26	69.22	27.63	13.36	6.98	4.11	417.82	393.37	24.45
3.5	148.57	71.54	30.81	15.23	8.14	4.8	427.66	399.49	28.17
4	148.63	73.8	33.75	16.6	9.31	5.49	436.21	404.81	31.4
4.5	148.71	76.03	36.55	18.06	10.47	6.17	444.7	410	34.7
5	148.77	77.65	38.38	19.38	11.38	6.86	451.19	413.57	37.62
5.5	148.81	77.71	39.17	20.68	12.16	7.54	454.88	414.5	40.38
6	148.84	77.48	39.95	21.97	12.93	8.23	458.24	415.11	43.13
6.5	148.86	77.52	40.74	23.26	13.63	8.85	461.72	415.98	45.74
7	148.88	77.61	41.71	24.46	14.28	9.3	465.12	417.08	48.04
7.5	148.9	77.61	42.1	25.74	14.98	9.75	467.98	417.51	50.47
8	148.93	77.61	42.7	26.75	15.68	10.2	470.8	418.17	52.63
8.55	148.95	77.62	43.2	27.2	16.38	10.65	472.95	418.72	54.23

Note: Total = 2*F+E+D+C+B+A
FED = 2*F+E+D
CBA = C+B+A
Forces in Kips, Displacement in inches

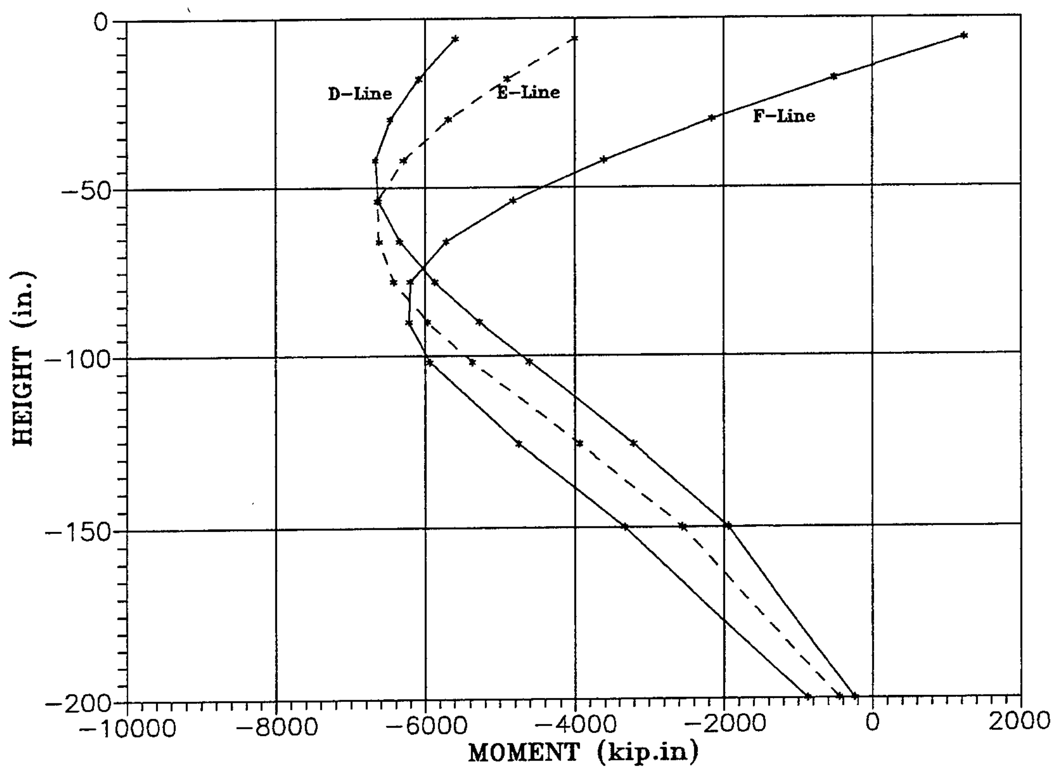


FIG. 13 MAXIMUM MOMENT PROFILES, F,E,D LINE PILES

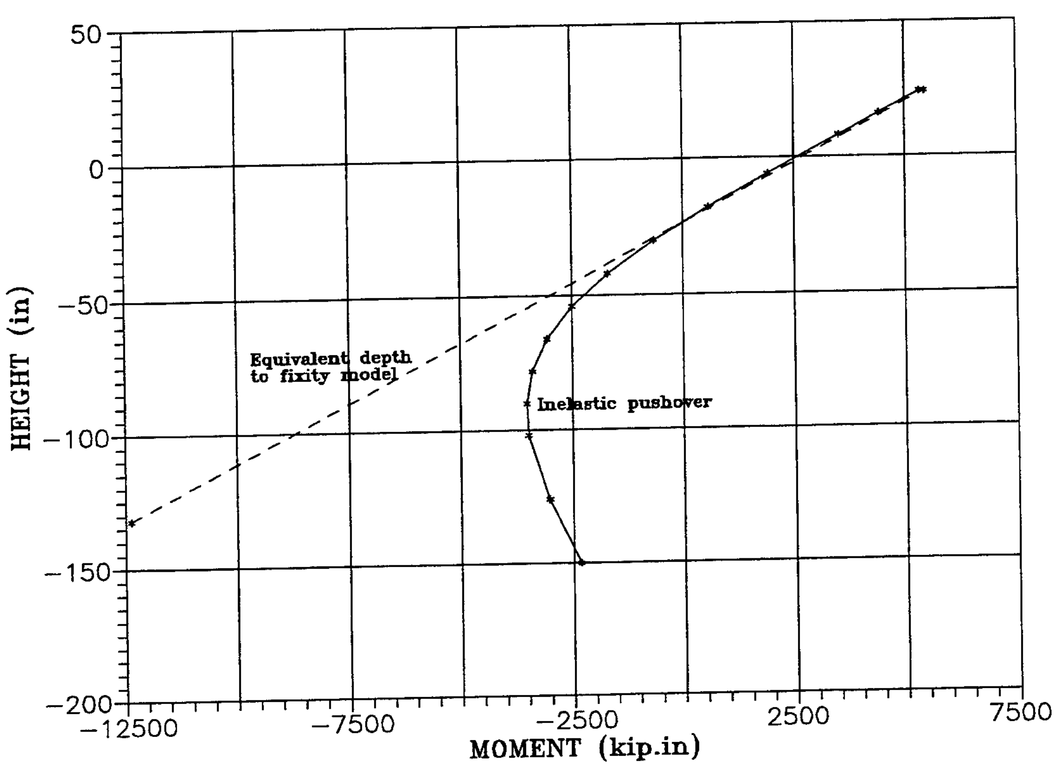


FIG. 14 F-LINE MOMENTS AT 0.94x YIELD DISPLACEMENT

maximum shear force developed in the longer piles is much less, and as a consequence, less depth of soil is required to resist the pile shear. This sort of information is only available from a full inelastic analysis, as carried out here.

Equivalent Fixity Model for Pushover: Section 4.5 of the calculations (pp C18 to C19) investigates the equivalent depth to fixity approach, and compares results with the more detailed inelastic pushover analysis. One of the problems with the depth to fixity approach is to know what value to use for the pile stiffness.

On pC18 two approaches for effective pile stiffness are tried. The first makes the approximation that the pile effective moment of inertia is equal to that of the dowel connection stiffness at first yield, over the full length of the pile. It is found that this results in an excessively flexible pile, with force-deflection stiffness only about half of that found from the more detailed pile pushover.

As shown on pC18, it is found that it is necessary to use variable moment of inertia up the height of the pile in order to accurately capture the elastic force-displacement stiffness. The dowel connection moment of inertia is adopted over the top 44in (plus the strain penetration length of 12in), with the remainder of the pile being given the uncracked section moment of inertia. This is appropriate since the pile doesn't crack below ground level before the dowel connection reaches yield moment. Equations are given on pC18 to determine the position of the point of contraflexure (distance l_t below the effective top of the pile), and hence, on pc19, to calculate the effective force-displacement stiffness, based on simple moment-area principles. A very close prediction of the actual stiffness results. Fig.14 compares predicted moment distributions with depth for the full inelastic push, and the variable-stiffness equivalent depth to fixity model at a displacement of 0.96in (94% of yield). It will be seen that very good agreement is obtained in the upper regions, but the moments in the region of the in-ground hinge are greatly overestimated by the equivalent depth to fixity model. Thus even if the depth at which the in-ground hinge develops can be estimated, the equivalent depth to fixity model will predict that the in-ground hinge will develop too soon. This would result in underestimating the displacement capacity based on the in-ground hinge strain limits.

Composite Pushover Response for 20ft Deck Segment: The pile pushover responses are summed in Table 2 to provide the composite force-displacement response for a 20ft segment of bridge, making due allowance for the increased number of piles on the F-Line. These data are also plotted in Fig. 15, together with a bi-linear approximation to the curve, to be used in later analyses.

In Fig. 16, and Table 2, the data in Fig. 15 are separated into two components, one representing the composite stiffness of the piles on the F, E, and D lines, and the other the composite stiffness of the piles on C, B, and A lines. Bi-linear approximations are also given for these curves. Data defining the bi-linear approximations are listed on pC33, and are needed for the inelastic time history analyses.

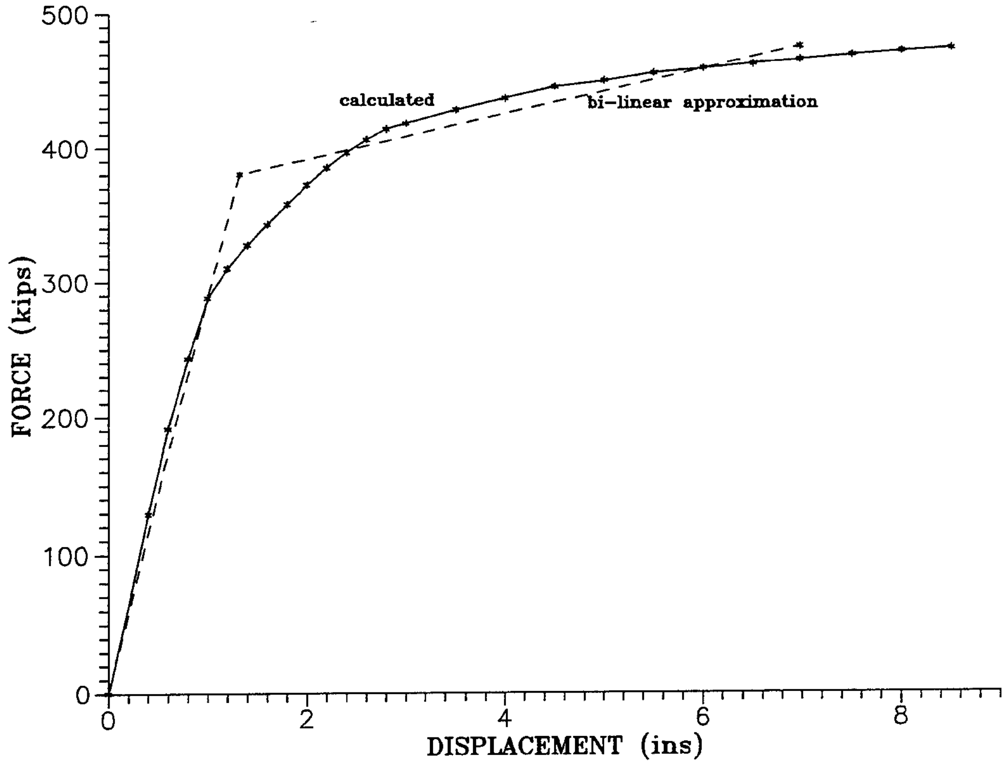


FIG. 15 TRANSVERSE FORCE-DISPLACEMENT RESPONSE (20ft segment

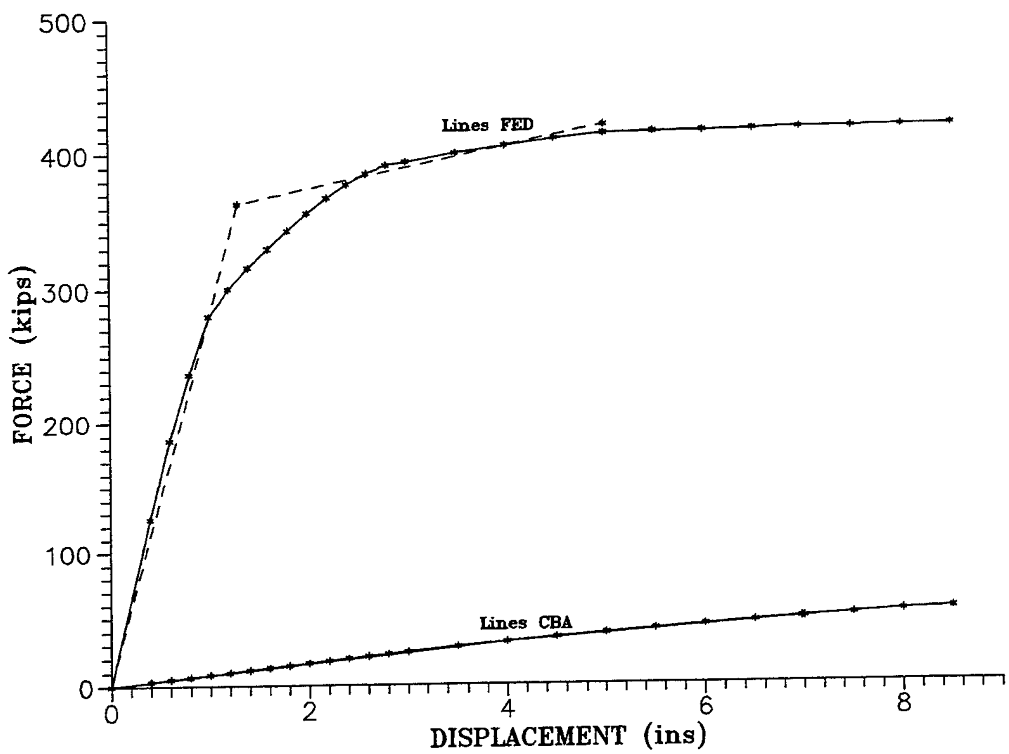


FIG. 16 TWO-PILE FORCE-DISPLACEMENT MODEL (20ft segment)

METHOD "A" ANALYSIS

Calculations for the single-degree-of-freedom Method A analysis are included in pages C21 to C23. This is a purely hand analysis approach, and very simple. An initial period of T = 0.602 sec is predicted, with a maximum displacement under pure transverse excitation of 3.78 in. Using the displacement amplification factor of Eqn 3-19 to account for torsional excitation and simultaneous longitudinal and transverse response, a maximum displacement of 5.23 in. is predicted. This is well within the capacity of the piles with W20 spirals @ 2.5 in centers, but above the capacity of the less well confined W11 @ 6in piles. Displacements under the Level 1 earthquake are found by direct scaling of the results for the Level 2 response, to give an estimate of 2.13 in. This is just acceptable for both pile designs.

It should be noted that the above assessment relates only to flexural response. It is possible that shear or joint failure could result in a lower assessed displacement capacity. This is in fact found to be the case for the W11 design.

METHOD "B" ANALYSIS

The method B multi mode analysis is carried out on a single wharf segment. As pointed out in the resource document, there is little point in carrying out a modal analysis of the linked wharves, as the nature of the interface between the segments cannot be adequately represented by an elastic analysis.

To simplify the analysis, the 400ft x 110ft segment, which contains 140 piles, is represented by an equivalent simplified structure containing 4 "super piles". These provide equivalent translational and torsional stiffness of the actual structure. Calculations on pp C24 and C25 show how the characteristics, and locations of the piles are defined.

The wharf segment is, of course, three dimensional. It is possible, however, to adequately describe it by a two-dimensional approximation, using a plan simulation. As shown in the sketch at the bottom of pC25, the four super-piles are represented by 2-D horizontal springs, providing the same lateral stiffness in transverse and longitudinal directions as the tributary piles of the super-pile. It can be argued that this may be adequate for determining the displacement response of the wharf, but will tell us nothing about the deck moments. This is true, but it must also be recognized that a full 3-D elastic analysis of the entire wharf segment, with all 140 piles separately modeled, and the deck represented by a 2-D grillage, or by a grid of shell elements, will also tell us nothing about the deck moments. The only way of finding these is from an inelastic analysis, and the easiest way to do this is to calculate the displacements at the pile tops from the simplified model described herein, and determine the corresponding pile top moments from results of the individual pile pushover analyses. These can then be applied as input moments to simpler models of the deck to determine the deck moments.

The model shown in pC25 includes stiff linking members between the super-piles to ensure effectively rigid in-plane behaviour of the deck.

The modal analysis of the 2-D simulation was carried out using Ruaumoko, though any dynamic analysis program would be suitable. Input data for the analysis is included in p A-26 of Appendix A, and the data echo and modal results from the analysis are listed in pp A-27 to A-30. The analysis indicates three significant modes. The first and third, with periods of 0.743sec and 0.503sec are longitudinal modes combining direct translation and torsion. Participation factors of 0.642 are calculated for both modes. The second mode, with T=0.602 sec has 100% participation in the transverse direction, and is identical to that calculated by the method A analysis.

Mode shapes for longitudinal response are shown on pC26. These refer to locations of the "super piles". On pC27, the necessary calculations to convert these to displacements at the corner piles of the wharf segments are noted, with the adjusted mode displacement components for the corner columns noted in the table at the top of pC28. Modal displacements, based on the response spectrum, modal period, and participation factor are also included in this table.

Some care is needed in the combination of these modal displacements. In the table on pC28, the typical combination, where the modal displacement components are combined by a SRSS rule is also listed. Note that, as the periods between the first and third mode are separated by about 40%, combinations based on a CQC approach will differ by less than 3% from these values. Also shown on this page are the necessary hand calculations for the 100%X +30%Y and 30%X+100%Y combinations. It will be noted that the case of 100% X + 30%Y results in peak vectorial displacements for the critical corner F-line pile of 6.27in, some 18% higher than for the Method A analysis.

This discrepancy does not indicate an error in the simplified method A analysis, but a basic, and common flaw in the method of modal combination adopted for Method B. The modal displacement shapes are shown in plan on pC28a. It will be noted that the displacement vectors of the corner pile identified as pile F for modes 1 and 3 are essentially orthogonal. Thus it is inconsistent to perform a SRSS combination on both X and y displacement components of the two modes and expect them to occur simultaneously, as was done in the combinations of pC28. In other words, if the X components of the two modes are added by a SRSS combination, the compatible combination of the Y components **must** be a square root **difference** of the squares. The necessary calculations, using a consistent combination, are carried out at the bottom of pC28a. The resulting maximum vectorial displacement for the corner pile on row F is now found to be 5.14 in. This is about 3% less than predicted by Method A, reflecting the conservative approach used to develop equation 3-19 which ignores torsional mass inertia.

It is thus concluded that, provided the modal combinations for Method B are carefully done, results will be very similar to those from Method A.

METHOD "C" ANALYSIS

The method C analysis is similar to the Method A analysis, but uses characteristic stiffnesses and damping at maximum response, rather than initial elastic values, using the substitute structure methodology. This requires a little iteration, since the final stiffness will not be known until the final displacement is known. The calculations on ppC29 to C32 show that these calculations are rather straightforward.

In the resource document, a fundamental approach is described for estimating the level of damping at maximum response. A simplified equation appropriate for Takeda hysteretic response (which best models pile inelastic response) in given at the bottom of pC29. A simple modification factor to reduce the elastic 5% response spectrum to the level appropriate for the calculated level of damping in given on pC30.

After 2 cycles of iteration, the peak displacement in the transverse direction is found to be 4.39 in. (16% higher than with the method A analysis). – see pC31. However, because of softening of the F-line and E-line piles, the center of stiffness appropriate at maximum displacement response is only 39.5ft from the center of mass, compared with 46.1ft for the initial stiffness calculations. This means that the amplification due to torsional response and combined X and Y components is less than for method A. As a consequence, the peak vectorial displacement for the corner F-line pile is estimated at 5.88 in, some 10.6% higher than for Method A.

Calculations are also given for the level 1 earthquake on pC32. This results in an answer only 4% different from the Method A prediction. The closer agreement is a result of reduced ductility at the serviceability limit state.

METHOD "D" ANALYSIS

A full inelastic analysis of the two linked wharf segments was carried out using the plan simulation described in the sketch on pC33 of the calculations. Essentially, this model incorporated two segments, each the same as used for the Method B analysis, linked by a central shear key. The "super-piles" were represented by bi-linear inelastic elements, using the relationships of Fig.16, defined on pC33. Hysteretic characteristics were based on the Takeda degrading stiffness model.

Normally, with interacting wharf segments, it is necessary to represent the interactions at the connecting movement joints with uni-directional impact springs at the corners to represent restraints impose by geometry as the movement joint tends to open and close, wedging at the corners. With two identical segments this is not necessary, as considerations of antisymmetry indicate that the two segments on opposite sides of the movement joints movement joints will always rotate an identical amount, thus completely avoiding impact.

The level of elastic viscous damping to be used requires careful consideration in an inelastic analysis. In most computer programs, this is largely related to the initial stiffness. This can greatly overestimate the effective damping at high levels of inelastic

response. To provide a reasonable simulation of the elastic damping assumed for the Method B analysis, which was 5% related to the secant stiffness to maximum response, an elastic damping level of 2% was chosen, since Ruaumoko adopted a Rayleigh damping (mixed initial stiffness and mass proportional damping).

Excitation was provided by scaling the 1940 El Centro record by a factor of 1.5. This provided a peak ground acceleration of about 0.5g in the NS component of the record, and reasonably good matching with the design spectrum in the critical 0.5 –1.0 sec. range. The model was subjected to three different analyses: pure transverse response to the NS component, pure longitudinal response to the NS component, and dual response to simultaneous excitation of the NS component in the longitudinal direction and the EW component in the transverse direction. The scaling factor of 1.5 was used for all components in all analyses.

Input data, and the data echo for the analyses are provided in Appendix A, pp A-31 to A-38.

Results of Analyses: Selected results of the analyses are shown in graphical form in Figs 17 to 25. Transverse response to 1.5xEl Centro NS, shown in Fig. 17, results in a peak displacement of 4.36 in. This compares with predicted peaks of 3.78 in from Methods A and B, and 4.27 in for Method C.

Response under longitudinal excitation is summarized in Figs 18 and 19. Note that the displacements plotted are at the "super-pile" locations, and need extrapolation to the corner piles to obtain peak values. Peak longitudinal response, extrapolated to the corner piles using the relationship developed on pC27 is 3.71in on Line F and 4.36 in. on Line A. Peak transverse displacements at the corners furthest away from the shear key, extrapolated from the Node 1 and Node 3 displacements of 2.132in and 0.566 in respectively are 2.712in. These are less than the Method B values based on conventional SRSS analysis of 4.595in (pC28) or on rational combination, of 3.324in (pC28a). The differences are largely due to the differences in geometry imposed by considering two linked wharf segments, rather than a single stand alone segment.

Maximum shear key force between the segments, shown in Fig.20, is 1801 kips. This is about 15% higher than predicted using Eqn 3-20.
Figures 21 and 22 plot the longitudinal and transverse response, respectively, when the two-segment model is subjected to simultaneous excitation in the longitudinal (El Centro NS x1.5) and transverse (El Centro EW x1.5) directions. Comparison of Figs 18 and 21 indicates **that in this case** longitudinal response is not increased by dual axis excitation. On the other hand, comparing figures 22 and 19, we see that transverse response is significantly increased, and occurs at a different time from the single axis case.

Shear key forces under dual excitation are plotted in Fig.23, and are found to be lower than for the single axis excitation (Fig.20). The peak values of about 1490 kips are very close to the prediction of Eqn.3-20.

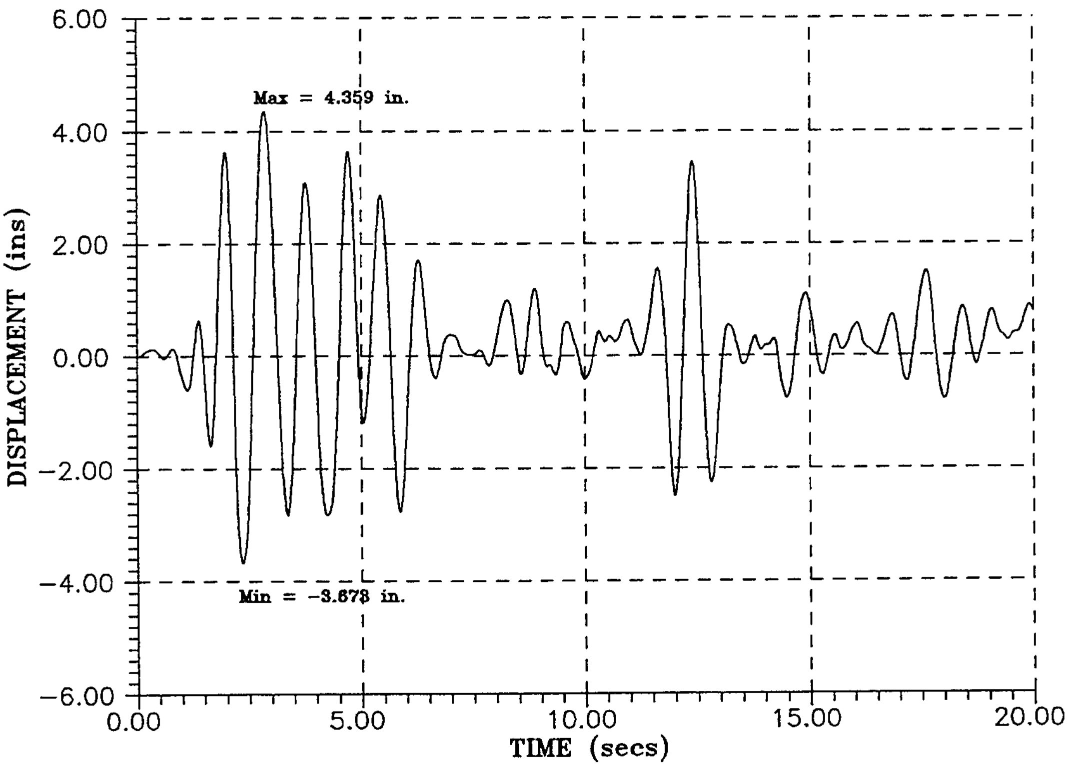


FIG. 17 TRANSVERSE EXCITATION, TRANSVERSE RESPONSE

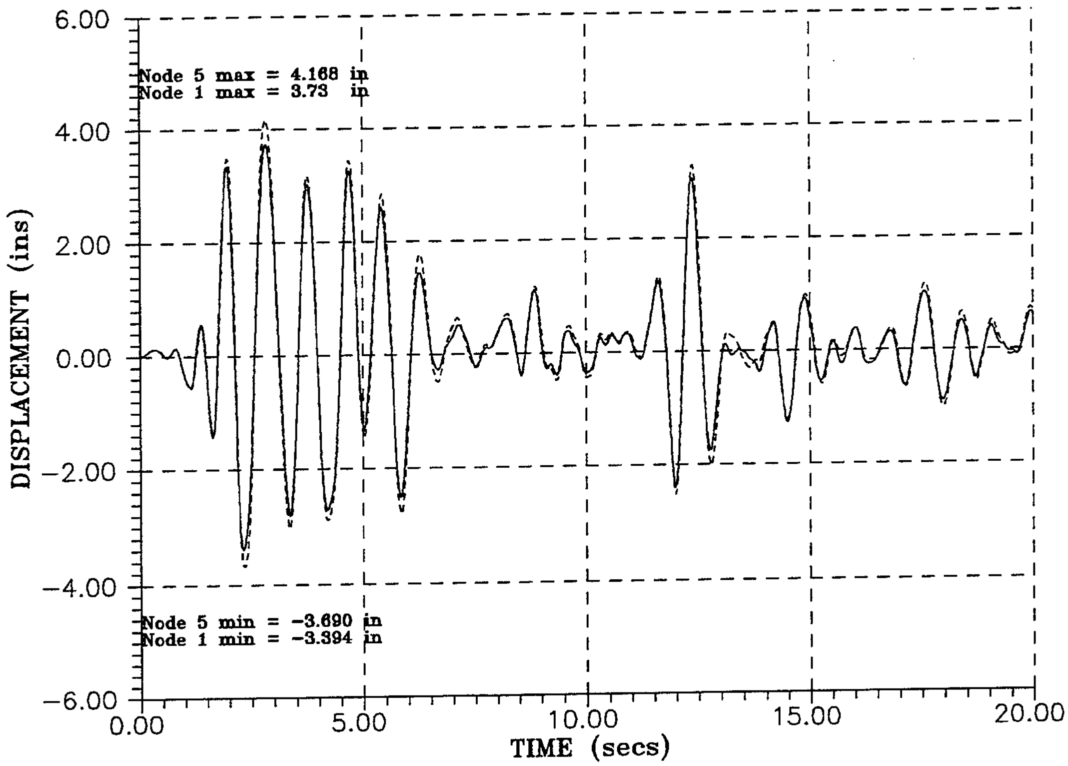


FIG.18 LONGITUDINAL EXCITATION AND RESPONSE (N1 and N5)

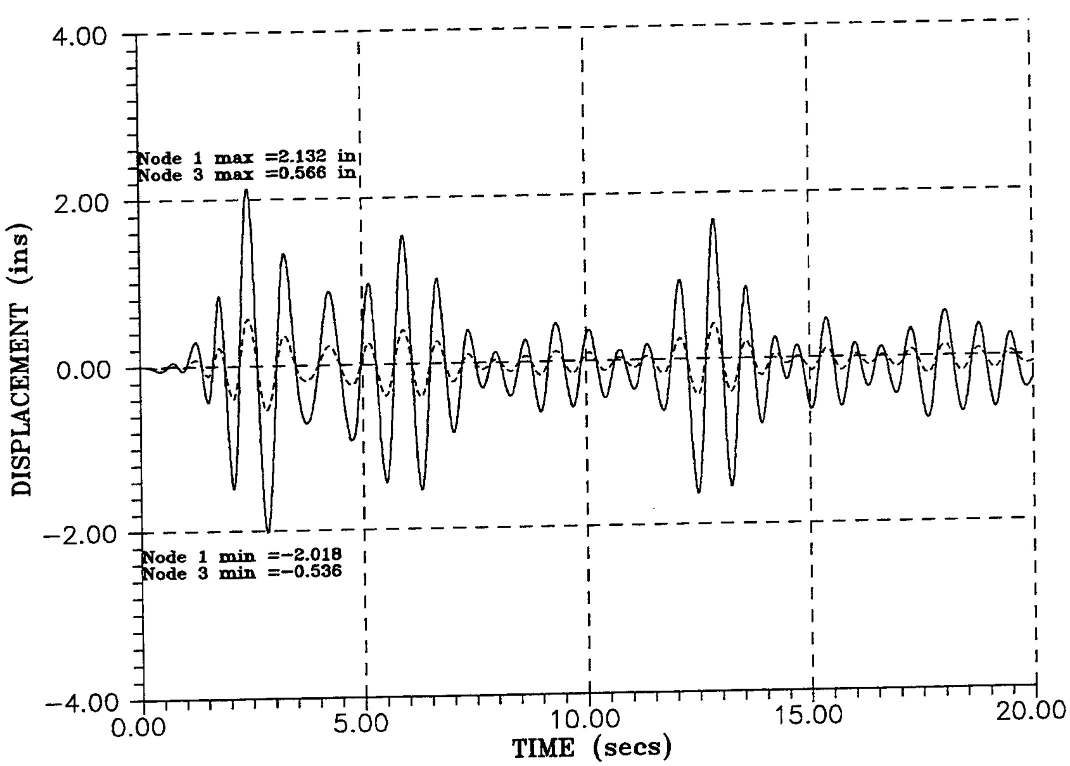


FIG.19 LONG. EXCITATION, TRANSVERSE RESPONSE (N 1,3)

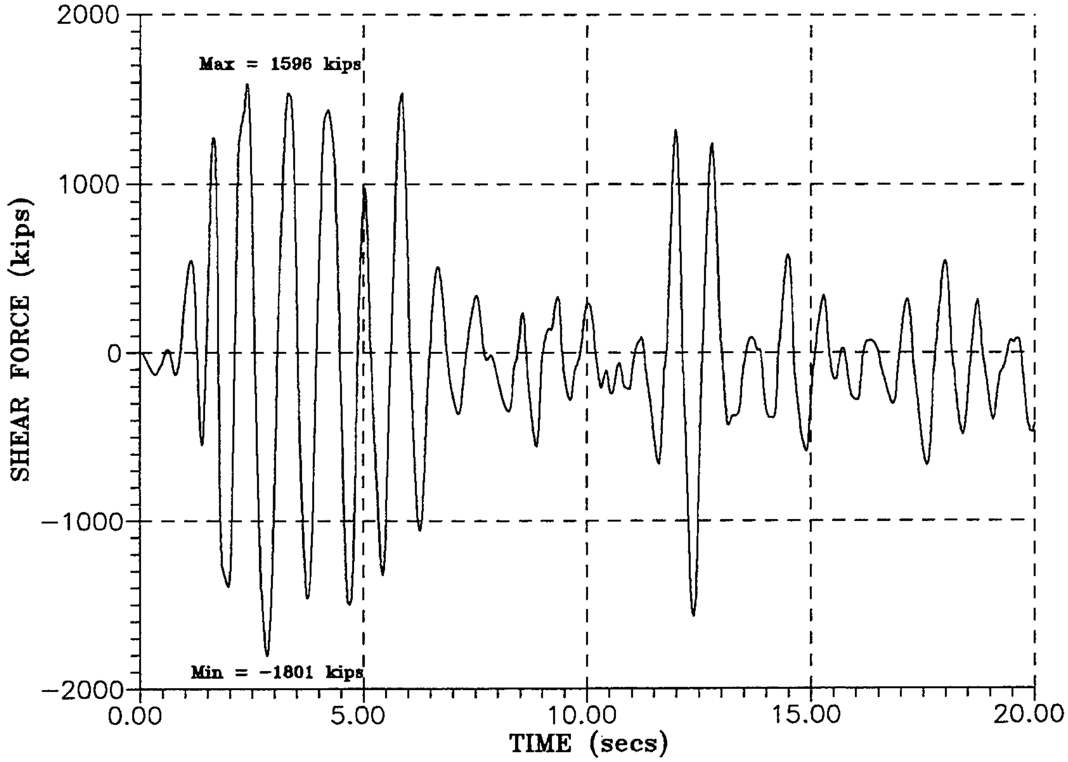


FIG. 20 LONGITUDINAL EXCITATION, SHEAR KEY FORCE

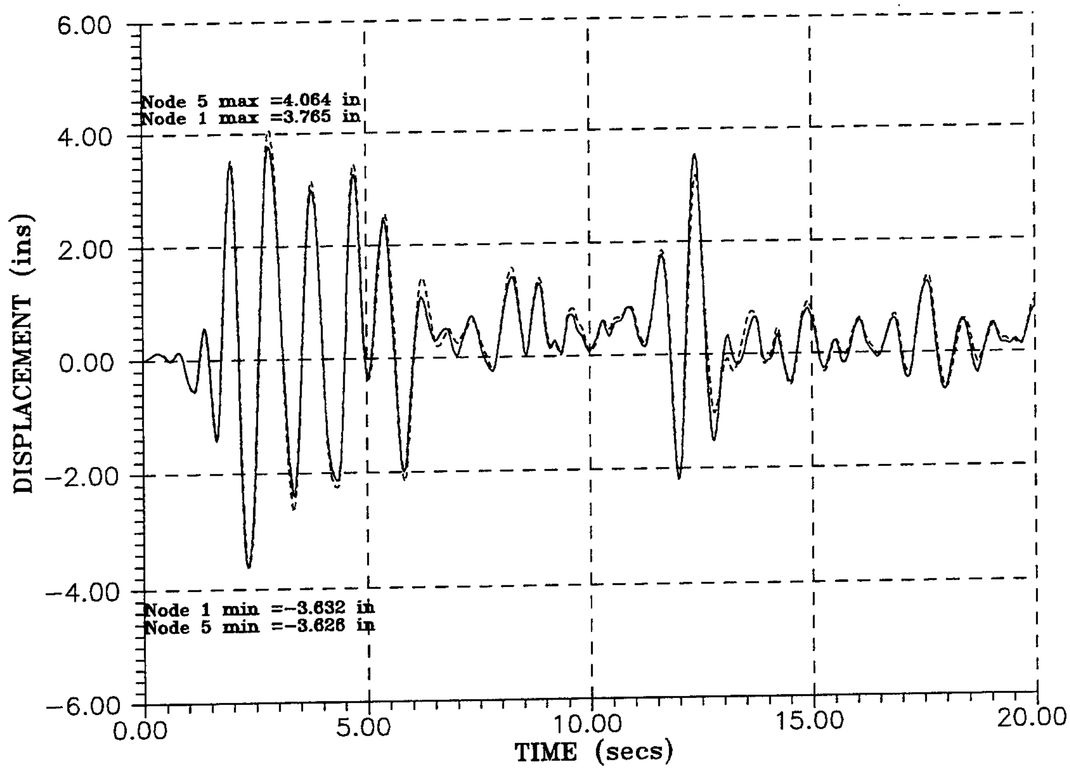


FIG.21 DUAL EXCITATION, LONGITUDINAL RESPONSE

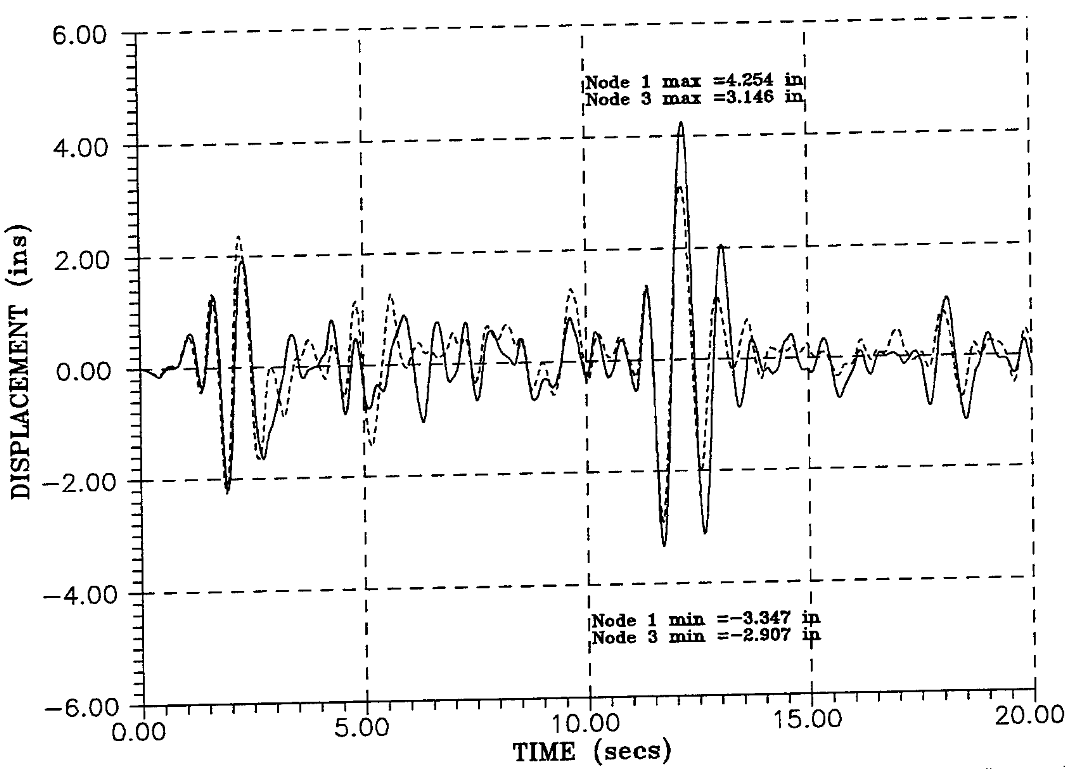


FIG.22 DUAL EXCITATION, TRANSVERSE RESPONSE

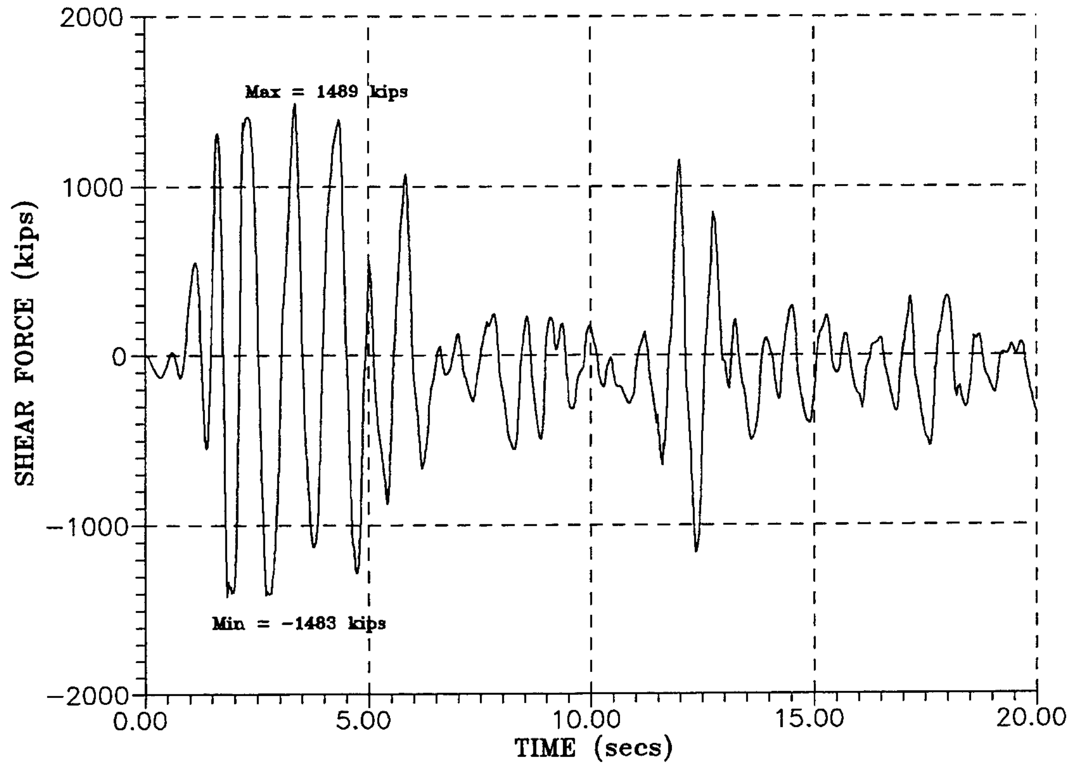


FIG.23 DUAL EXCITATION, SHEAR KEY FORCE

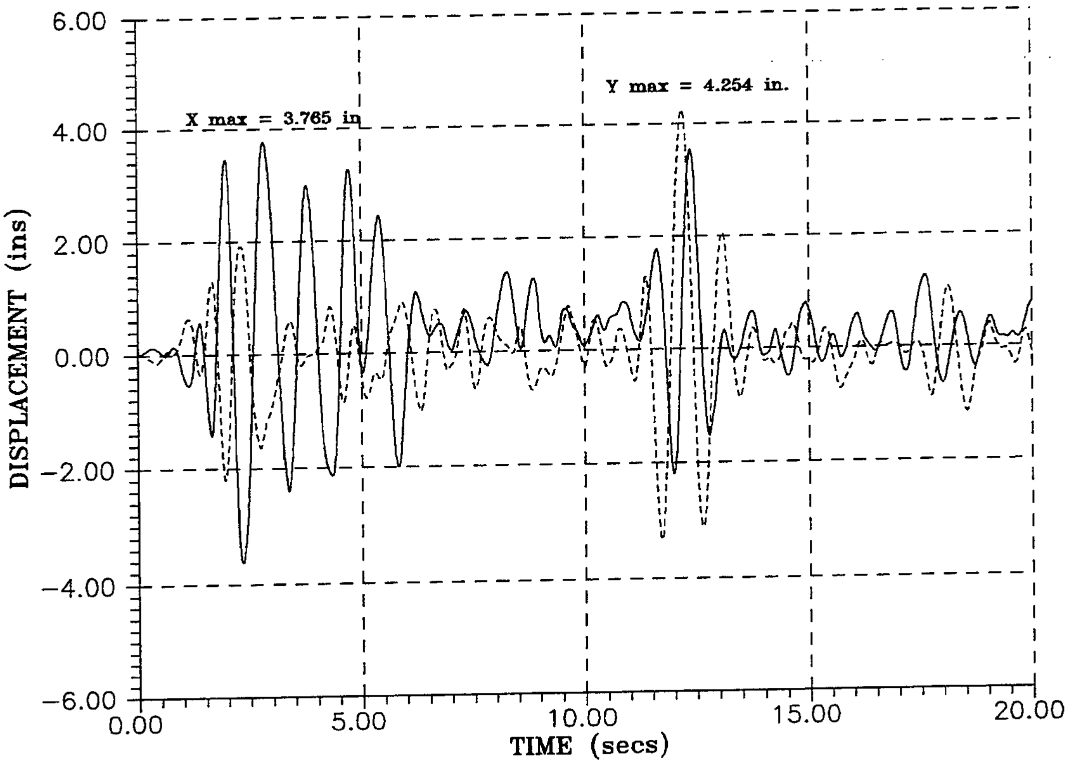


FIG.24 DUAL EXCITATION, NODE 1 DISPLACEMENT COMPONENTS

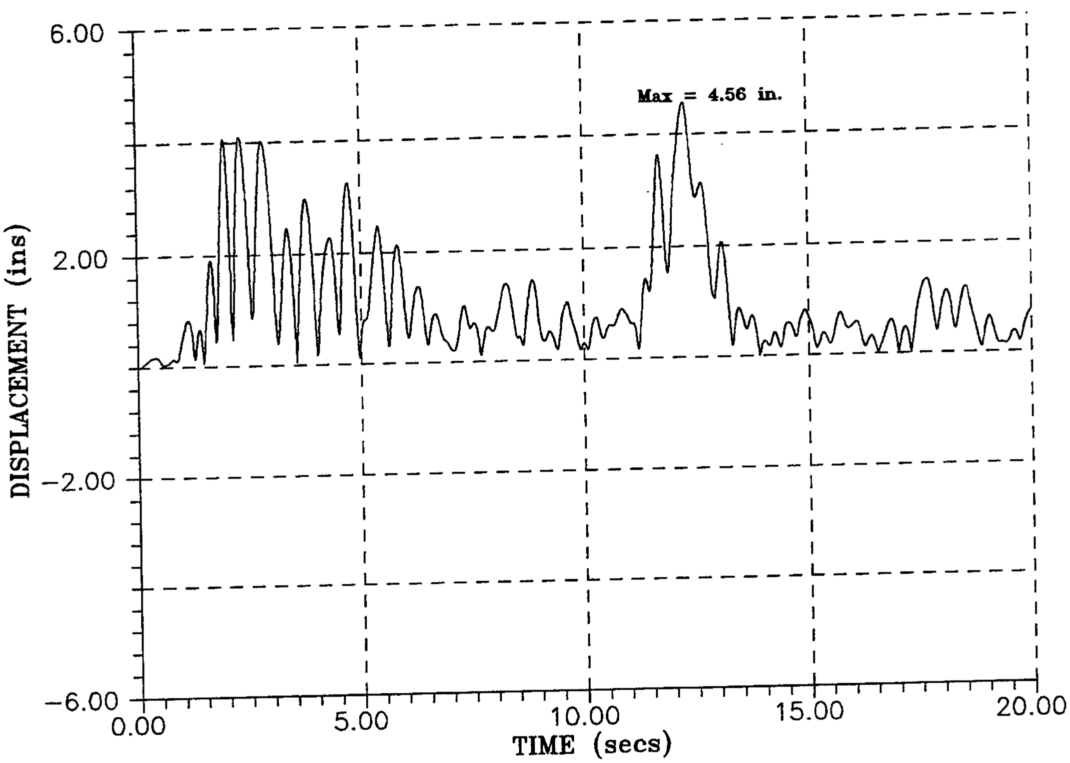


FIG.25 DUAL EXCITATION, NODE 1 VECTOR DISPLACEMENT MAGNITUDE

It is also of interest to combine the X and Y components of response to obtain peak vectorial displacements. Fig.24 plots longitudinal (solid line) and transverse (dashed line) for the same node under dual axis excitation. It will be noted that the peak longitudinal and transverse response tend not to coincide in time. The final figure (Fig. 25) plots the absolute magnitude of Node 1 displacement, found from the SRSS of the longitudinal and transverse components in Fig. 24. All displacements are thus positive, and no information is provided about the vectorial direction of response. However, it is of interest that the peak vectorial displacement, of 4.56 in., is only 7% larger than the peak Y axis response of Fig.24, and is much less than any of the predictions of the simplified models. Not too much should be read into this result, as considerable differences could occur if a different pair of earthquake records had been used as excitation, but it perhaps illustrates the advantages of using the more sophisticated Method D analysis.

PILE SHEAR STRENGTH

Calculations comparing pile shear strength and capacity are included in pp C36 to C40. Separate calculations are provided for the two different cases of spiral confinement (W20 @ 2.5in and W11 @ 6in).

Shear demand is found from the calculated maximum displacement demand, and the pile pushover analyses. Since the F-line piles are critical, only this line is considered. Conservatively, the calculated peak shear force is amplified by a factor of 1.4, as suggested in the resource document, to cope with the possibility of material strengths exceeding design values, and soil strength also exceeding design values. The latter is a very significant possibility, since it would force the in-ground plastic hinge to form higher in the pile, thus increasing the pile shear force.

Shear strength is found from the combination of concrete, truss, and axial force contributions. The first requires that the curvature ductility be calculated, as shown on p37. Since the critical plastic hinge forms at the top of the pile, it would be unwise to consider the beneficial influence of prestress in the axial force contribution.

Results from the calculations indicate that the pile confined with W20 @ 2.5 in. has adequate shear strength, but that shear failure of the less well confined W11 @ 6in is certain before the displacement **capacity** is reached. Recall that the displacement capacity corresponding to flexural failure was 3.63in for this pile, and was less than the displacement demand. On p40 calculations are included to show how the displacement at shear failure of the W11 pile can be estimated. The best-estimate is for a displacement of 2.05in, though shear failure could occur at as low a displacement as 1.05 in., if shear demand is at the maximum feasible value.

CONNECTION STRENGTH

Calculations to determine the performance of the two connection details illustrated in Fig.3 are included in ppC41 to C43. In both cases it is found that the anchorage length is satisfactory (though if checked by ACI rules, both would fail). Joint shear strength for the detail with special joint shear reinforcement is found to be satisfactory, and no joint shear problems will occur. The detail of Fig. 3(b) is found to be marginal, but joint shear failure is likely as the ductility increases. Calculations to estimate the degradation of moment capacity at the pile top with increasing joint rotation are included on pC43. Note that in this case the potential for joint failure is less serious than the pile shear failure, which would dominate the assessment conclusion.

SEISMIC ASSESSMENT OF A TWO-SEGMENT MARGINAL WHARF

CALCULATIONS APPENDIX

DESIGN CALCULATIONS

1. STRUCTURE MASS : Consider a tributary length of 20ft.

Deck weight : $(2.75 \times 110 \times 20) \times 0.15\text{k/cuft} = 907.5$ kips

Live load : $(110 \times 20) \times 0.035\text{ k/sq ft} = 77$ kips
(piping & equipment)

984.5 kips

Pile weight : $0.15 \times \pi \times 1^2 = 0.471$ kips/ft

Effective tributary weight at deck level = 33% of pile to depth of 10ft below top of rip-rap. Again consider 20ft deck length, and note line F has 2 piles/20 ft. Refer Fig 1

Line F :	$0.333 \times (2+10) \times 0.471 (\times 2)$	= 3.76 kips
E :	$0.333 \times (10.57+10) \times 0.471$	= 3.23 kips
D :	$0.333 \times (22+10) \times 0.471$	= 5.02 kips
C :	$0.333 \times (33.43+10) \times 0.471$	= 6.81 kips
B :	$0.333 \times (44.86+10) \times 0.471$	= 8.60 kips
A :	$0.333 \times (56.29+10) \times 0.471$	= 10.40 kips
		37.82 kips

TOTAL WEIGHT = 984.5 + 37.8 = **1022.3 kips**

2. Center of Mass : $\bar{x}$ from landward edge.

$$\bar{x} = \frac{(984.5 \times 55) + (3.8 \times 3.5) + (3.2 \times 23.5) + (5.0 \times 43.5) + (6.8 \times 63.5) + (8.6 \times 83.5) + (10.4 \times 103.5)}{1022.3}$$

$= \mathbf{55.44\,ft}$

3. PILE MOMENT-CURVATURE CHARACTERISTICS

3.1 Axial Force Levels (DL plus LL)

Typical pile : $20\text{ft} \times 20\text{ft} \times [2.75 \times 0.15 + 0.035] = 179\text{ kips}$

Line A : approx $20 \times 16.5 \times [2.75 \times 0.15 + 0.035] = 148\text{ kips}$

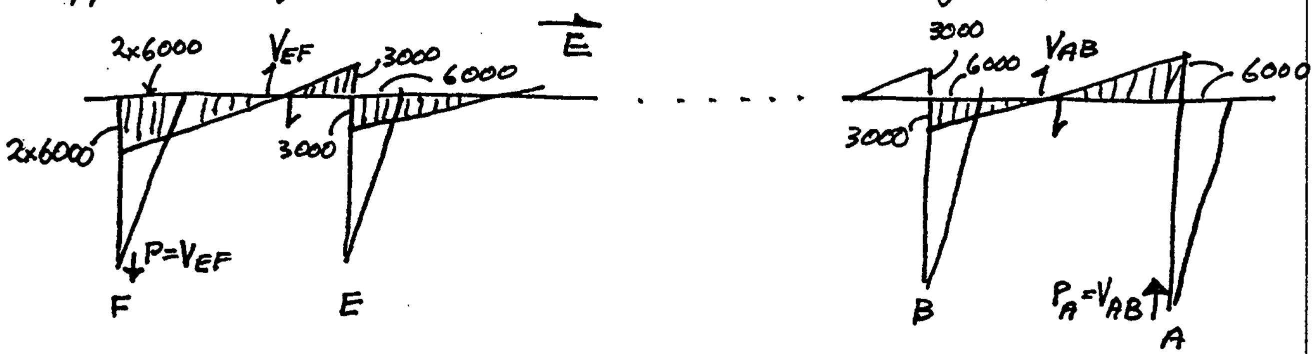
Line F : approx $13.5 \times 10 \times [\quad " \quad] = 60\text{ kips}$.

Maximum weight of pile (for force at in-ground hinge) - assume @ 2D

Line A : $0.471 \times (56.29 + 4) = 28.4\text{ kips}$.

Seismic Axial force.

Initial analyses indicate a connection moment capacity of approximately 6000 kip.ins. Estimate seismic axial forces from deck shears:



$$P_A = V_{AB} = \frac{3000+6000}{20 \times 12} = 37.5\text{ kips}$$

$$P_F = V_{EF} = -\frac{1}{2}\left(\frac{12000+3000}{20 \times 12}\right) = -31.3\text{ kips}$$

Seismic forces on other piles are lower.

Thus : Max pile force $\approx 148 + 28 + 38 = 214\text{ kips}$

Min. " " $\approx 60 - 31 = 29\text{ kips}$.

Analyze piles for $P = 0, 100, 200$ kips. (Note - unlikely A will form hinge)

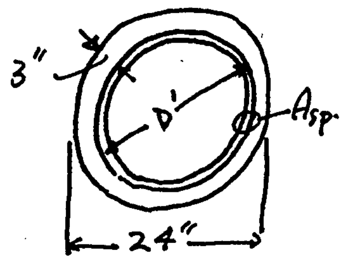
3.2 Ultimate Concrete Strain capacity

Eqn (3-26), p3-61 : $\varepsilon_{cu} = 0.004 + 1.4\,\rho_s f_{yh}\,\varepsilon_{sm}/f'_{cc} \geq 0.005$

(a) Spiral = W20 @ 2.5"

$A_{sp} = 0.20\,in^2$
$s = 2.5\,in.$

$\rho_s = \dfrac{4A_{sp}}{D's}$



$D' = 24 - 2\times3 - 0.5$

$= 17.5\,in.$

$\therefore\ \rho_s = \dfrac{4\times0.2}{17.5\times2.5}$

$= 0.0183$

For W20, A82, take $\varepsilon_{sm} = 0.10$, $f'_{cc} \approx 1.5\times7.0 = 10.5\,ksi.$

$\therefore\ \varepsilon_{cu} = 0.004 + \dfrac{1.4\times0.0183\times70\times0.1}{10.5}$

ie $\underline{\underline{\varepsilon_{cu} = 0.021}}$

(This is less than the maximum permitted level of 0.025 [p 3-62]

(b) spiral = W11 @ 6.0"

$D' = 24'' - 6'' - 0.375''$

$= 17.63''$

$\rho_s = \dfrac{4\times0.11}{17.63\times6}$

$= 0.00416$

$\varepsilon_{cu} = 0.004 + \dfrac{1.4\times0.00416\times70\times0.10}{10.5}$

ie $\underline{\underline{\varepsilon_{cu} = 0.00788}}$

• For the pile/deck Dowel connection, the above compression strains apply. For the in-ground hinge a maximum value of 0.008 applies, and would govern for the W20 @ 2.5 in. case. Note that this is to avoid damage requiring repair below ground. If the design criterion is "no collapse" the full compression strain capacity of 0.021 could be used.

3.3 Analyses for Moment-Curvature

• Moment curvature analyses for the prestressed sections were carried out using CIRPRESS. Input data is shown in Appendix A, pp A1 & A2. Note that the prestressing tendons are located in pairs at the same heights (distances) y from the section tension each. The area of 0.43 in^2 is the area of two tendons. The program is designed for dual mild steel plus prestressing steel, and a dummy mild steel area of 0.01 in^2 has been used.

Output for the W20 @ 2.5" cases for P=100, 0, 200 kips are listed on pp A3 to A5. N.A. depth = neutral axis depth from compression edge, tendon stress & strain refer to extreme tension tendon, moment and curvature are in kip.in, & in^{-1} respectively.

• Data for the W11 @ 6.0 cases are identical to the W20 @ 2.5 in (p A1) with modification only to tie diameter (0.374 in) and tie spacing (6.0 in). Moment curvature results are tabulated on pp A6 to A8 for P = 100, 0, 200 kips respectively.

• Data for the pile/deck dowel connection are in p A9 for W20 @ 2.5", and in p A16 for W11 @ 6". Moment curvature

listings are in pp A10–A14 and A17–A19 for W20@2.5" and W11@6" respectively.

• Note that the moment-curvature curves continue to strains higher than the ultimate compression strains calculated on p3 above, as a somewhat less conservative estimation of ultimate strain is used in the program.

3.4 Cracking Moment and Curvature

• For the prestressed piles, the first data point calculated corresponds to an extreme fiber compression strain of 0.001. It is useful to calculate the moment and curvature corresponding to cracking:

Section modulus: $Z = \frac{\pi D^3}{32} = \frac{\pi \times 24^3}{32} = 1357 in^3$

Tension strength: $f'_t = 7.5\sqrt{7000} = 627.5 psi$

Prestress : $f_p = 1172 psi$ after losses

Axial compression: $P = 100 \rightarrow 221 psi$

$P = 200 \rightarrow 442 psi$.

Modulus of Elasticity: $E_c = 57000\sqrt{7000 psi} = 4770 ksi$.

$P = 0$ $f = 627.5 + 1172 = \underline{1800 psi}$ $M_{cr} = fZ = \underline{2443 kip\ in}$

$P = 100$ $f = 1800 + 221 = 2021 psi$ $M_{cr} = \underline{2742 kip\ in}$

$P = 200$ $f = 1800 + 442 = 2242 psi$ $M_{cr} = \underline{3042 kip\ in}$

$P = 0$ $\varepsilon = f/E_c = 377.4 \times 10^{-6}$ $\phi_{cr} = \varepsilon/(D/2) = 31.5 \times 10^{-6}/in$

$P = 100$ $\varepsilon = 423.7 \times 10^{-6}$ $\phi_{cr} = 35.3 \times 10^{-6}/in$

$P = 200$ $\varepsilon = 470.0 \times 10^{-6}$ $\phi_{cr} = 39.2 \times 10^{-6}/in$

3.5 Moment-Curvature Summary

Moment-curvature curves are plotted in the main body of the report. A summary of the moments and curvatures corresponding to serviceability and damage control limit states is listed below.

Serviceability limit states are based on

$\varepsilon_c = 0.004$

$\varepsilon_s = 0.010$

$\Delta\varepsilon_p = 0.005 \Rightarrow \varepsilon_p = 0.0053 + 0.005 = 0.0103$

$\boxed{\text{LIMIT STATES MOMENTS + CURVATURES}}$

(a) Prestressed Section

Confinement	P (kips)	Serviceability M_s kip.in	ϕ_s 1/in	Damage Control M_u kip.in	ϕ_u 1/in.
W20@2.5"	0	6112	0.000403	5723	0.002664
	100	6536	0.000422	6132	0.002519
	200	6881	0.000419	6513	0.002389
W11@6"	0	6113	0.000401	5216	0.000915
	100	6533	0.000422	5513	0.000849
	200	6866	0.000419	5757	0.000790

(b) Dowel Section

Confinement	P (kips)	Serviceability M_s kip.in	ϕ_s 1/in	Damage Control M_u kip.in	ϕ_u 1/in.
W20@2.5"	0	~~0.~~5540	0.000672	5642	0.003048
	100	6087	0.000614	6011	0.002729
	200	6620	0.000564	6356	0.002590
W11@6"	0	5537	0.000672	5350	0.001195
	100	6083	0.000615	5732	0.001102
	200	6615	0.000565	6077	0.001022

• Note that serviceability moments & curvatures are not significantly influenced by amount of confinement, but damage control values are.

• Note also that moments + curvatures for prestressed section, W20@2.5" are for $\varepsilon_c = 0.021$. If the in-ground limit of 0.008 is applied, the moments + curvatures for W11@6" may be used without significant error, since $\varepsilon_c = 0.00788$ for this case.

4. PILE PUSH OVER ANALYSES

4.1 Moment-curvature Approximation for Push Analyses

• In order to carry out pushover analyses it is necessary to simplify the analytical curves of Figs 5-8.

4.1.1 • Pile/Deck Dowel Connection.

Over the development length of the prestressing strand (say top 4ft) the pile characteristics will be governed by the dowel connection.

F line piles: Axial force may vary between 30 + 90 Kips (see p C2) From Table on p. C6 the variation in strength will be less than 2% from the mean value (P=60 kips) – use this for all directions of seismic response.

Spalling causes reduction to strength which is largely regained at high curvatures. Use an elasto-plastic approximation, with strength = 97% of serviceability moment.

Elastic Stiffness: Dowel yield strain $\varepsilon_y = 66/29000 = 0.002276$

for $\underline{P=100}$ (p A10) at $\varepsilon_s = 0.002255$, $M = 4671$ kip.in, $\phi = 0.0001842$/in

$$\therefore E_c I_{eff} = M/\phi = \frac{4671}{184.2} \times 10^6 \text{ kip in}^2$$

$$= 25.36 \times 10^6 \text{ kip in}^2$$

Since $E_c = 4770$ksi, $I_{eff} = \dfrac{25.36 \times 10^6}{4770}$

$$= \underline{5316 \text{ in}^4}$$

<u>Note</u> $I_{gross} = \dfrac{\pi D^4}{64} = \dfrac{\pi \times 24^4}{64}$

$$= 16286 \text{ in}^4$$

$$\therefore I_{eff} = \frac{5316}{16286} = 0.326\, I_{gross}$$

for P = 0 (p A12) at $\varepsilon_s = 0.002238$, $M = 3948$ $\phi = 0.0001733$/in

$$\therefore E_c I_{eff} = \frac{3948}{173.3} \times 10^6$$

$$= 22.78 \times 10^6 \text{ kip in}^2$$

$$I_{eff} = \frac{22.78 \times 10^6}{4770}$$

$$= \underline{4776 \text{ in}^4} = 0.313\, I_{gross}$$

<u>Interpolate for P = 60 kips</u> $\rightarrow$ $I_{eff} = \underline{\underline{5100 \text{ in}^4}}$

<u>Strength</u> $\left.\begin{array}{l} P = 100 \rightarrow M_s = 6087 \text{ kip in} \\ P = 0 \rightarrow M_s = 5540 \text{ kip in} \end{array}\right\}$ p C6

$\therefore P = 60 \rightarrow M_s = 5868$ kip in.

Design strength for pushover $= 0.97 M_s = 0.97 \times 5868 = \underline{5692 \text{ kip in}}$

• Note: The above procedure, applied for P = 100 kips is plotted as an elasto-plastic approx. to the actual moment-curvature data in Fig 9.

Piles on Lines A–E (dowel top connection)

P = 180 kips (Note: also used for A, though axial force is lower, since significance of pile A to pushover response is v. low – see later results).

for P = 200, (pA14) at $\varepsilon_s = 0.002281$, M = 5377, $\phi = 0.0001956/in$

$$\therefore E_c I_{eff} = \frac{5377}{195.6} \times 10^6 \ kip\,in^2$$

$$= 27.49 \times 10^6 \ kip.in^2$$

$$\therefore I_{eff} = \frac{27.49 \times 10^6}{4770}$$

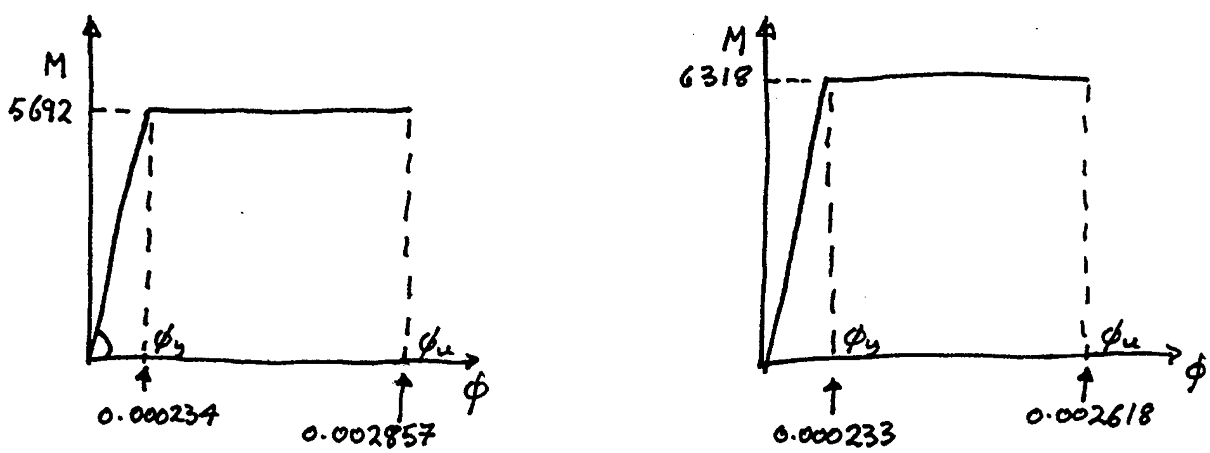
$$= \underline{5763 \ in^4} = 0.354 I_{gross}$$

Interpolate for P = 180 kips → I_{eff} = 5674 in² (0.348 I_{gross})

Strength: P = 200, → M_s = 6620 kip in

∴ P = 180 → M_s = 6513 kip.in

Design strength for pushover = 0.97 M_s = 0.97 × 6513 = 6318 kip in.



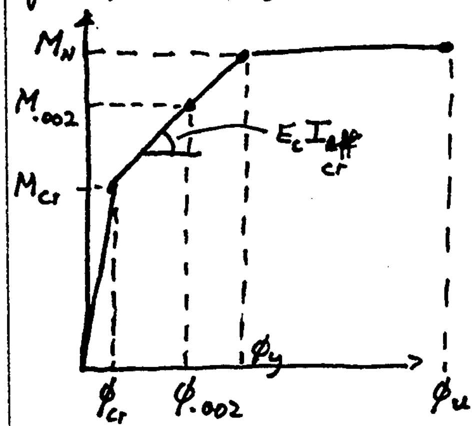
F Line A–E Lines

PILE TOP MOM-CURVATURE APPROX

(Note: $\phi_y = \frac{M}{E_c I_{eff}}$ in plots above. e.g for F Line $\phi_y = \frac{5692}{4770 \times 5100} = 0.000234$)

4.1.2 Prestressed Pile Sections

For pile sections lower than 4ft below the deck, use the prestressed pile sections. Since these crack at rather high moments, use a tri-linear (Muto) approximation. Increase axial force for pile weight: P= 70kips, line F, 200kips, lines A–E.



- M_{cr}, ϕ_{cr} from p C5
- Use moment $M_{.002}$ and curvature $\phi_{.002}$ at compression strain of $\varepsilon_c = 0.002$ to characterize cracked-section stiffness (from Tables, p A3–A5)
- $M_N = 0.97 \times M_s$ (p.C6)
- ϕu from p.C6.

(a) F line : $P = 70$ kips , $I_{gross} = 16286\,in^4$

$\underline{M_{cr} = 2650\,kip\,in}$ $\quad \underline{\phi_{cr} = 0.0000342/in.}$ (p C5).

at $P = 100$ (p A3) $M_{.002} = 5953$, $\phi_{.002} = 0.000193$

$$E_c I_{cr} = \frac{M_{.002} - M_{cr}}{\phi_{.002} - \phi_{cr}} = \left(\frac{5953 - 2742}{193 - 35.3}\right) \times 10^6\ kip\,in^2$$

$$= 20.36 \times 10^6\ kip\,in^2$$

$$\therefore I_{cr} = \frac{20.36 \times 10^6}{4770}$$

$$= \underline{4268\,in^4} \quad = \underline{0.262\,I_{gross}}$$

$P = 0$ (p A4) $M_{.002} = 5647$ $\quad \phi_{.002} = 0.000212$

$$\therefore E_c I_{cr} = \left(\frac{5647 - 2443}{212 - 31.5}\right) \times 10^6 \text{ kip.in}$$

$$= 17.75 \times 10^6 \text{ kip in}$$

$$\therefore I_{cr} = \frac{17.75 \times 10^6}{4770}$$

$$= \underline{3721 \text{ in}^4} = \underline{0.228 I_{gross}}$$

<u>Interpolate for P = 70 kips</u>

$$I_{cr} = \underline{4104 \text{ in}^4} = \underline{0.252 I_{gross}}$$

from pC6 $\quad M_s = 6409$ kip in

$$\therefore M_N = 0.97 M_s = \underline{6217 \text{ kip in}} \qquad \phi_y = \left(\cancel{31.5} \times 10^{-6} + \frac{6217 - 2650}{4770 \times 4103}\right) = \underline{0.0002165/\text{in.}}$$

Ultimate: $M_N = \underline{6217}$, $\phi_u = \underline{0.002563}$ (interpolated, pC6).

(b) <u>Lines A–E</u> : P = 200 kips.

$$\underline{M_{cr} = 3042 \text{ kip in}} \qquad \underline{\phi_{cr} = 39.2 \times 10^{-6}/\text{in}} \qquad (\text{pC5})$$

at P=200 (pA5) $\quad M_{.002} = 6171$ kip.in. $\quad \phi_{.002} = 0.000178/\text{in}$

$$\therefore EI_{cr} = \frac{(6171 - 3042)}{(178 - 39.2)} \times 10^6 \text{ kip in}^2$$

$$= 22.54 \times 10^6 \qquad (I_{eff} = 4725 \text{ in}^4)$$

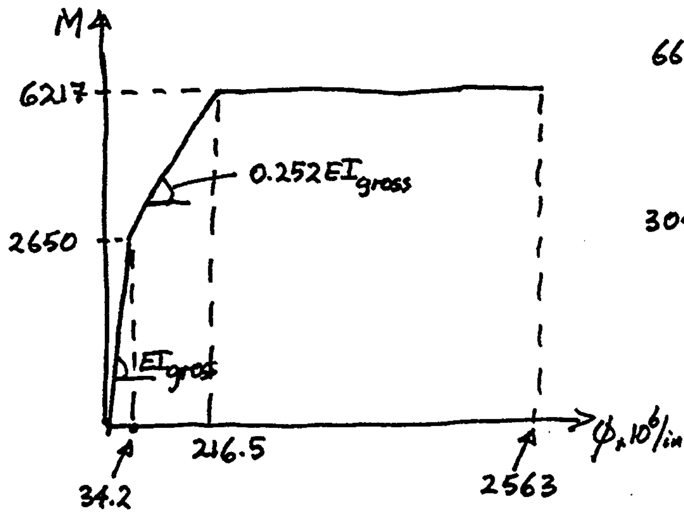
$$= 0.290\ EI_{gross}$$

$$M_N = 0.97 \times 6881 = \underline{6675 \text{ kip in}}$$

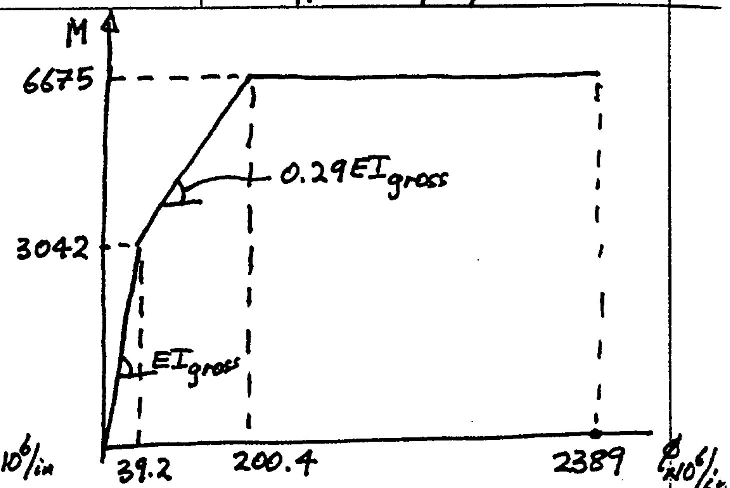
$$\phi_y = 39.2 \times 10^{-6} + \frac{(6675 - 3042)}{4770 \times 4725}$$

$$= \underline{200.4 \times 10^{-6}/\text{in}}$$

$$M_u = M_N = 6675, \qquad \phi_u = 2389 \times 10^{-6}/\text{in.}$$



F line



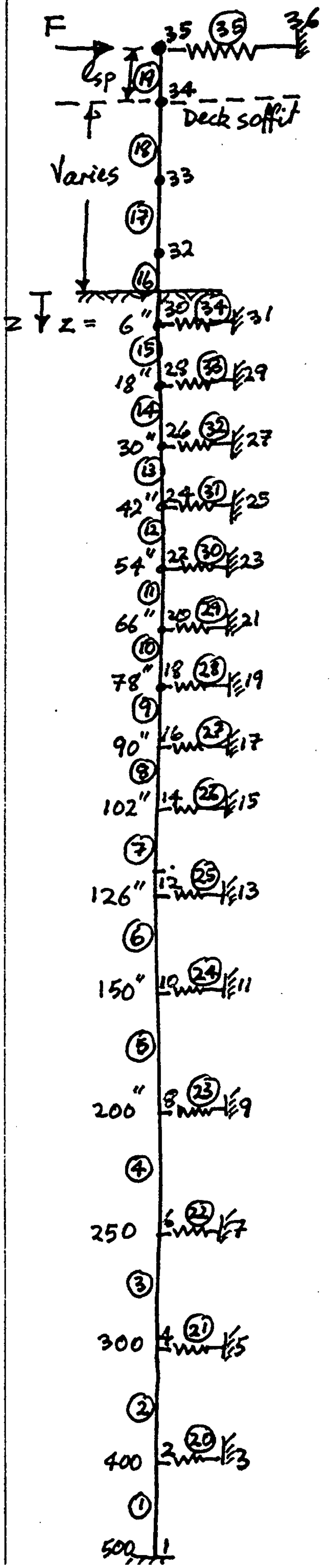
A-E lines

PRESTRESSED SECTIONS MOMENT-CURVATURE APPROXIMATION

- The Muto tri-linear approx. for lines A-E above is plotted together with the P = 200 kips data in Fig 10. Note that the discrepancy will be less than apparent in this Figure due to soil confinement delaying spalling, and comparatively low curvatures at maximum response.

4.2 SPRING MODEL FOR PUSHOVER ANALYSES

- The pushover analysis is carried out on a pile-by-pile basis, as suggested on pp 3-~~2~~54 and Fig 3-23. Initial calculations show that even for the short F line piles, assuming an effective depth to fixity of 10 ft below ground surface, and allowing for cracking, the deck stiffness is more than 30 times the pile stiffness. As a consequence, deck flexibility will have an insignificant effect on response, and the deck may be assumed to be rigid.
- The most realistic structural model includes inelastic soil springs. The model adopted is sketched on pC13,



Notes
- Fixity assumed 500" below ground
- Nodes shown thus: 1 to 36
- Members shown thus: 1 to 35
- Top node (35) is fixed against rotation, and is located $l_{sp} = 0.15 f_y d_{bl} = 0.15 \times 66 \times 1.27'' = \underline{\underline{12.6''}}$ above deck soffit, to account for strain penetration.
- Members 20 to 34 represent soil springs
- Member 35 is an artificial spring of high stiffness acting in parallel with the pile to ensure positive stiffness when a full mechanism develops in the pile.
- Program used is Ruaumoko (an inelastic time-history program) with a slowly ramping acceleration, to simulate the increasing inertia force.
- Data for the F line pile is included in Appendix A.
- Pile properties as per pp C9 and C12 (Dowel properties to ground, or top 4ft only).
- Soil spring properties next page.

4.3 Spring properties for Level ground

MEMBER ~~SPRING~~	HEIGHT (Z)	p_{ult}	ΔH	F_{ULT}	k	SPRING NO.
34	-6"	0.25 k/in	12"	3.0 kips	6.0 kip/in	18
33	-18"	0.75	12	9.0	18.0	17
32	-30	1.25	12	15.0	30.0	16
31	-42	1.75	12	21.0	42.0	15
30	-54	2.25	12	27.0	54.0	14
29	-66	2.75	12	33.0	66.0	13
28	-78	3.25	12	39.0	78.0	12
27	-90	3.75	12	45.0	90.0	11
26	-102	4.25	18	76.5	153	10
25	-126	5.25	24	126.0	252	9
24	-150	6.25	37	231.3	463	8
23	-200	8.33	50	416.5	833	7
22	-250	10.42	50	521.0	1042	6 ELASTIC
21	-300	12.5	75	937.5	1875	5
20	-400	16.67	100	1667	3333	4

Note: $p_{ult} = 0.5z$ kip/in , z in ft $\therefore p_{ult} = 0.0417z$, z in inches

ΔH = tributary pile height for spring

$F_{ULT} = p_{ULT} \cdot \Delta H$ = ultimate strength of spring (Yield force) at 0.5"

$k = F_{ULT}/0.5$ = elastic spring stiffness.

MEMBERS 20-24 modeled as elastic springs ($\Delta < 0.5$ in)

MEMBERS 25-34 " " inelastic springs.

Nodal heights for above-ground nodes are as follows:

Node:	F line	E line	D line	C line	B line	A line
32	8	40	108	176	245	315
33	16	78.8	216	353.2	490.3	627.5
34	24	126.8	264	401.2	538.3	675.5
35,36	36.6	139.4	276.6	413.8	550.9	688.1

4.4 RESULTS OF PILE PUSHOVER

Results of the individual pile pushover analyses are expressed in terms of pile shear force vs deck displacement in Fig 11, to a maximum displacement of 8.55 in, which is the calculated displacement capacity of the F line piles as established below.

4.4.1 F-line displacement Limits

From the F line pushover, the pile top yield moment of $M_N = 5692$ kip.in. occurs at a displacement of $\underline{1.027 in} = \Delta y$. A second plastic hinge, 90 in. below grade develops in the pile at a displacement of 2.737 in. [Distance between hinges = 90+24 = 114 in]

Plastic hinge length

pile top: Eqn 3-30(b): $L_p = 0.08L + 0.15 f_y d_b \geq 0.30 f_y d_b$

$$L = (90+24)/2 = 57 in.$$

$$f_y = 66 ksi$$

$$d_b = 1.27 in.$$

$$\therefore\ L_p = 0.08 \times 57 + 0.15 \times 66 \times 1.27 = 17.13 in \geq 0.30 \times 66 \times 1.27 = \underline{\underline{25.15 in}}$$

$\therefore L_p = 25.15$ in. governs

<u>In-ground hinge</u>: From Fig 3-27, with dimensionless parameter ≈ 5 take $L_p \approx 1.8 D_p = 1.8 \times 24 = 43.2$ in. [Note: the loco moment gradient from the push-over analysis indicates this will be conservative – see Fig 12]

<u>Displacement Limits Based on Pile-top Hinge:</u>

	W20 @ 2.5 in	W11 @ 6 in.
ϕ_y (p C9)	234×10^{-6}/in	234×10^{-6}/in.
<u>Serviceability</u>		
ϕ_s (p C6)	637×10^{-6}/in	637×10^{-6}/in
$\theta_{ps} = (\phi_s - \phi_y) L_p$ (Eqn 3-29)	$(637-234) \times 10^{-6} \times 25.15$	
i.e θ_{ps}	$= 0.01014$	0.01014
$\Delta_s = \Delta_y + \theta_{ps} (90+24)$	$1.027 + 0.01014 \times 114$	
ie Δ_s	= <u>2.18 in.</u>	= <u>2.18 in.</u>
<u>Damage-control</u>		
ϕ_u (p C6, p C9)	2857×10^{-6}/in	1139×10^{-6}/in
$\theta_{pu} = (\phi_u - \phi_y) L_p$ (Eqn 3-29)	$(2857-234) \times 10^{-6} \times 25.15$ $= 0.0660$	$(1139-234) \times 10^{-6} \times 25.15$ $= 0.0228$
$\Delta_u = \Delta_y + \theta_{pu} (90+24)$	$1.027 + 0.066 \times 114$ = <u>8.55 in.</u>	$1.027 + 0.0228 \times 114$ = <u>3.63 in.</u>

Displacement Limits Based on In-ground Hinge:

	W20 @ 2.5in	W11 @ 6in.
ϕ_y (pC12)	216.5×10^{-6}/in	216.5×10^{-6}/in
<u>Serviceability:</u>		
ϕ_s (pC6, interpolated)	416×10^{-6}/in	416×10^{-6}/in.
$\theta_{ps} = (\phi_s - \phi_y)L_p$ (Eqn 3-29)	$(416-216.5) \times 10^{-6} \times 43.2$	
i.e θ_{ps}	$= 0.00862$	$= 0.00862$
$\Delta_s = \Delta_y + \theta_{ps} \times 114''$	$2.737 + 0.00862 \times 114$	
i.e Δ_s	$=$ <u>3.72in.</u>	<u>3.72in.</u>
<u>Damage Control</u>		
ϕ_u (pC6, pC12)	2563×10^{-6}/in	869×10^{-6}/in
$\theta_{pu} = (\phi_u - \phi_y)L_p$	$(2563-216.5) \times 43.2 \times 10^{-6}$	$(869-216.5) \times 10^{-6} \times 43.2$
	$= 0.1014$	0.0282
$\Delta_u = \Delta_y + \theta_{pu} \times 114''$	$2.737 + 0.1014 \times 114$	$2.737 + 0.0282 \times 114$
i.e Δ_u	$=$ <u>14.30in.</u>*	$=$ <u>5.95 in.</u>

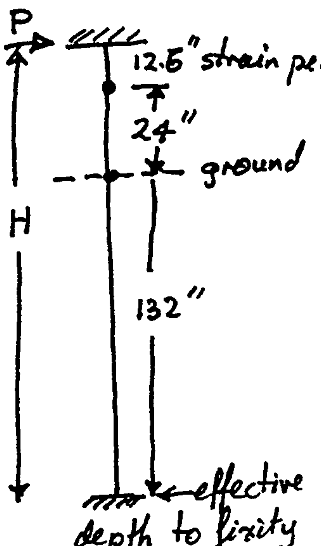
*Note: If in-ground concrete compression strain limit of 0.008 is applied, Δ_u reduces to 5.95in (see p C7).

Note that there is no need to calculate the displacement limits for piles other than the F-row, since these will significantly exceed those for the F-row.

4.5 PUSHOVER BASED ON EQUIVALENT DEPTH TO FIXITY.

Chapter 3, and Figs 3-20 + 3-21 discuss usage of equivalent depth to fixity for lateral stiffness calculations. From Fig 3-21(b), with a typical value for the dimensionless parameter of 5, the effective depth to fixity is about $5.5D_p$ for F line, and $4.7D_p$ for A line.

F-line Pile:



Question: what to use for effective stiffness of pile at first yield?

(a) If based on dowell connection stiffness,

$$EI_{eff} = 23.9\times10^6 \text{ kip in}^2 \qquad (\text{p } C8)$$

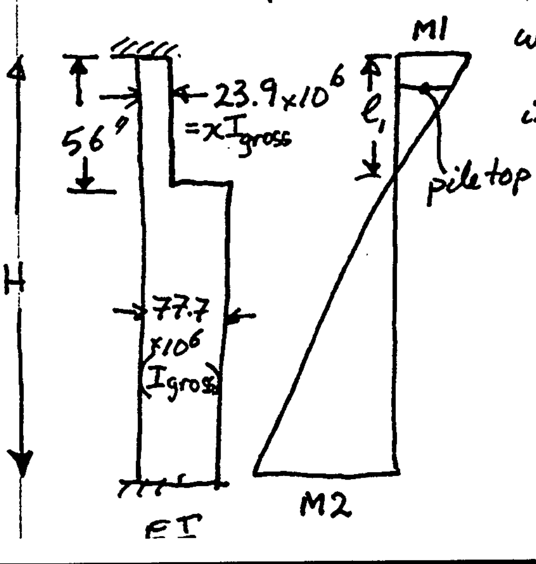
$$\text{Then } K = \frac{P}{\Delta} = \frac{12EI_{eff}}{H^3} = \frac{12\times23.9\times10^6}{168.7^3}$$

$$= 59.7 \text{ kip/in}$$

Detailed inelastic pushover (Fig 11) for F-pile gives $K =$ 116.7 kip/in at yield.

∴ model based on dowel connection stiffness is too flexible

(b) Use $EI_{eff} = 23.9\times10^6$ for top 44" + strain penetration [say 56" total] with EI_{gross} below, since pile barely cracks in-ground at pile-top first yield.



$$l_1 = \frac{xH^2 + 3136k}{112k + 2xH} \qquad x = \frac{23.9}{77.7} = 0.308$$

$$k = 1 - x = 0.692 \qquad \left(x = \frac{I_{eff}}{I_{gross}}\right)$$

$$= \frac{0.308\times168.7^2 + 3136\times0.692}{112\times0.692 + 2\times0.308\times168.7}$$

$$= \underline{60.3 \text{ in.}}$$

$M_2 = \left[\frac{168.7-60.3}{60.3}\right] M_1 = 1.80 M_1$, $M_1 = 60.3P$

$$\Delta \doteq \frac{60.3P \times 60.3^2}{3 \times 23.9 \times 10^6} + \frac{1.8 \times 60.3P \times 108.4^2}{3 \times 77.7 \times 10^6}$$

$$= 0.00306P + 0.00547P$$

$$= 0.00853P$$

$\therefore K = \frac{P}{\Delta} = \underline{117.2\ \text{kip/in}}$. ← Very close to inelastic pushover. (p C18).

Thus the effective depth to fixity model can give good representation of stiffness, if EI variation is modeled.

Check moments:

From inelastic pushover, at $\Delta = 0.96$ in (94% of yield), $P = \underline{114.0\ \text{kips}}$.

Equivalent depth to fixity: $M_1 = 60.3 \times 114 = 6874$ kip.in. [~~cf. 5328~~]

$M_2 = -1.8 \times 6874 = -12{,}373$ kip.in.

Expected in-ground max. moment location = −90" [from inelastic pushover, Fig 12]

∴ predicted moment at −90" is

$$M_{-90} = 6874 - (90+24+12.6) \times 114$$
$$= -7558 \text{ kip in.}$$

predicted moment at top of pile (12.6" below M_1)

is $M_{top} = 6874 - 12.6 \times 114$

$= \underline{5438\ \text{kip.in.}}$ cf $\underline{5328}$ from In. Push

Thus: good agreement for pile top, but poor agreement at in-ground location. Cannot use equivalent depth to fixity for inelastic response of piles with short ground clearance.

4.6 COMPOSITE PUSHOVER ENVELOPE

Results from the individual pile pushover analyses are combined in Table 2 and Fig.15 to produce the structural pushover force-displacement response appropriate to transverse excitation related to a 20ft longitudinal tributary length. Since there are 2 F-line piles/20ft length and 1 E, D, C, B, A line piles, the composite resistance is found as

$$F_{20,\Delta} = F_{A,\Delta} + F_{B,\Delta} + F_{C,\Delta} + F_{D,\Delta} + F_{E,\Delta} + 2F_{F,\Delta}$$

where $F_{A,\Delta}$ is the resistance of pile A at displacement Δ, etc.

Also shown in Fig 15 is a bi-linear approximation intended to provide best representation of the predicted response up to 7.0 in. displacement. Initial estimates indicated response would be less than this. Coordinates of the bi-linear approximation are

F	Δ
0	0
380 kips	1.32 in
474.4	6.97 in

5 RESPONSE ANALYSIS

Calculate Response transversely + longitudinally using different levels of analysis.

5.1 METHOD A ANALYSIS: EQUIVALENT SINGLE MODE ANALYSIS

5.1.1 Pure Transverse Analysis

Refer pp 3-52 and 3-53.

From bilinear approximation (p C20, Fig 15) at yield, initial stiffness is

$K_i = 380/1.32 = 288$ kips/in / 20 ft length.

Tributary weight $W = 1022.3$ kips/20 ft (p C1)

Period: $T = 2\pi\sqrt{\dfrac{W}{gK_i}}$

$= 2\pi\sqrt{\dfrac{1022.3}{386 \times 288}}$

$= \underline{0.602 \text{ sec.}}$

From Site specific acceleration response spectrum (Table 1)

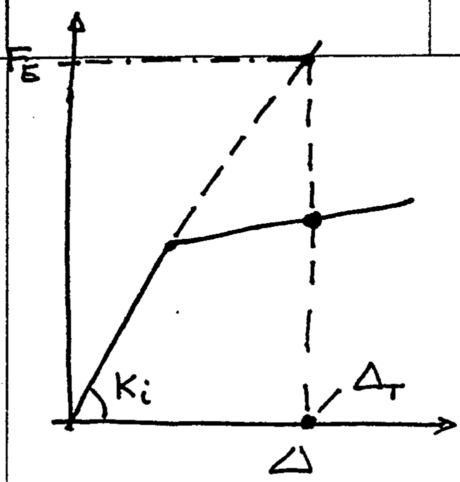
Response acceleration (elastic) $= 1.064g$.

$\therefore$ Elastic response force $= 1.064 \times 1022.3 = F_E$

$F_E = 1088$ kips / 20 ft length.

Note that this is not the true response force, but the elastic response for a structure with $T = 0.602$ sec, and unlimited elastic strength.

Using the equal displacement approximation, displacement of the inelastic system = displacement of the elastic system (next page)



F

$$\therefore \Delta_T = \frac{F_E}{K_i} = \frac{1088}{288}$$

$$= \underline{3.78 \text{ in.}}$$

Thus the Method A estimate of displacement under pure transverse excitation is $\underline{\underline{\Delta_T = 3.78 \text{ in.}}}$

5.1.2 <u>Amplification for combined Transverse + Longitudinal Response.</u>

Calculate elastic eccentricity under longitudinal response.

Eqn 3-22: $$x_T = \frac{\Sigma n_i V_i x_i}{\Sigma n_i V_i}$$

where n_i = number of piles / tributary length on row i

V_i = shear force in each pile on row i at specified displacement

x_i = distance of row i from datum.

Chose row F as datum, and $\Delta = 1.0$ in (98% of yield) as specified displacement

From table 2:

$$x_T = \frac{(2 \times 117.4 \times 0) + (34.7 \times 20) + (10.4 \times 40) + (4.45 \times 60) + (2.23 \times 80) + (1.37 \times 100)}{288}$$

$= \underline{5.88 \text{ ft.}}$ (from row F).

$\therefore$ Eccentricity between center of mass + center of stiffness is

$e = 55.44 - (5.88 + 3.5)$ [refer p. C1]

$= \underline{\underline{46.06 \text{ ft}}}$

Thus, from Eqn 3-19, for combined longitudinal and transverse response,

$$\Delta_{max} = \Delta_t \sqrt{1 + (0.3(1 + 20e/L_L))^2}$$

where L_L = Length of wharf segment = 400 ft

$$\therefore \Delta_{max} = 3.78\sqrt{(1 + (0.3(1 + 20 \times 46.06/400))^2}$$

$$= 3.78 \times 1.41$$

$$= 5.32 \text{ in.}$$

Assessment: Displacement capacity for W20@2.5" = 8.55 in (p C16) OK

Displacement capacity for W11 @ 6" = 3.63 in < 5.32 No good!

Level 1 Earthquake [Serviceability]

With Level 1 EQ = 40% of Level 2, Method A estimate of maximum displacement is

$$\Delta_{s\,max} = 0.40 \times 5.32 \text{ in}$$

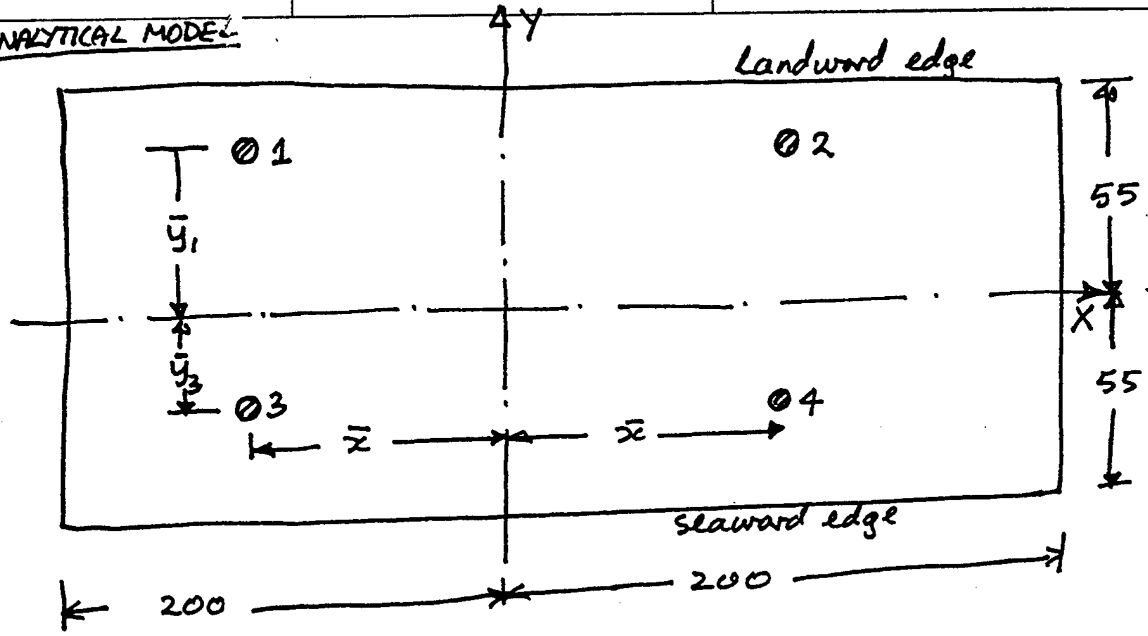
$$= \underline{2.13 \text{ in.}}$$

Assessment: Displacement capacity = 2.18 in, both cases [p C16] – just OK!

5.2 METHOD B ANALYSIS — MULTI-MODE RESPONSE

The multi-mode Method B analysis is found by investigating a single wharf segment [400 × 110 ft], representing the piles by 4 "super piles" of equivalent total translational and torsional stiffness. [piles 1,2,3,4 next page].

5.2.1 ANALYTICAL MODEL



Let Piles 1 + 2 each represent 200ft tributary length of F, E and D lines.
Let Piles 3 + 4 each represent 200 ft tributary length of C, B and A lines.

For correct torsional modeling, $\bar{x} = \frac{400}{\sqrt{12}} = \underline{\underline{115.5 ft}}$ (radius of gyration)

Location of superpiles in y direction

Calculate pile relative stiffnesses at 1.0 in. displacement [ref Table 2]
[+Fig 1]

Superpile 1: Eccentricity from F line:

$$e_{1F} = \frac{(20 \times 34.7) + (40 \times 10.37)}{(2 \times 117.4 + 34.7 + 10.37)} = \frac{1109}{279.9} = \underline{\underline{3.96'}}$$

$$\therefore \bar{y}_1 = 55 - (3.96 + 3.5) = 47.54'$$

Superpile 3: Eccentricity from C line:

$$e_{3C} = \frac{(20 \times 2.33) + (40 \times 1.37)}{(4.45 + 2.33 + 1.37)} = \frac{101.4}{8.15} = \underline{\underline{12.44'}}$$

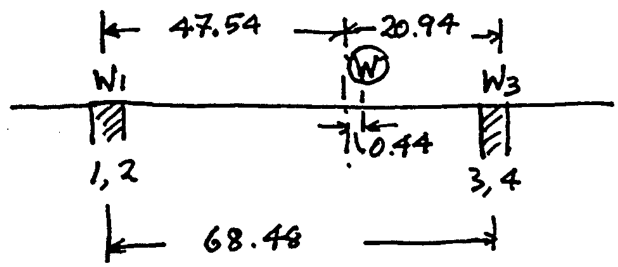
$$\therefore \bar{y}_3 = 8.5 + 12.44 = \underline{\underline{20.94'}}$$

Superpile Stiffness:

Piles 1,2: $K_1 = 279.9$ kip/in/20ft × 10 = 2800 kip/in. (Refer Table 2, FED @ 1")

Piles 3,4: $K_3 = 8.15$ kip/in/20ft × 10 = 81.5 kip/in. (Refer Table 2, CBA @ 1")

Weight: Distribute to top of superpiles 1,2,3,4. Note center of weight is 55.44 ft from landward edge [i.e. $y_{mass} = -0.44$ ft]
∴ Weight must be distributed to reflect this:



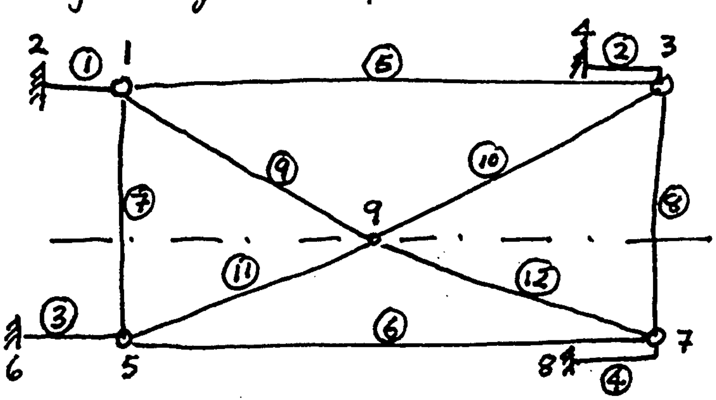
W = 1022.3 × 20 = 20446 kips.

$W_{1,2} = 0.5 \times 20446 (20.94 - 0.44)/68.48 = \underline{3061 \text{ kips}}$

$W_{3,4} = 0.5 \times 20446 (47.54 + 0.44)/68.48 = \underline{7163 \text{ kips}}$

5.2.2 ANALYSIS

Any dynamic analysis program could be used. I have used Ruaumoko for the modal analysis, since I will also use it for the time-history analysis. A plan view is modeled:



Notes: Nodes shown thus: 1 to 9
- Members shown thus: 1 to 12
- Members 1 to 4 are 2-D springs (X+Y direction + θ) representing superpiles
- Members 5 to 12 are stiff to force diaphragm action of deck.

Data Listing is included in Appendix A.

5.2.3 ANALYSIS OUTPUT

Results of the data echo/interpretation, and modal analysis are included in sub-Appendix A, pp A-B-2 to A-B-5. Modes 1 and 3 (p A-B-5) represent the combined translation/torsion in the longitudinal (x) direction, and mode 2 the uniform transverse (Y) mode. The other modes are due to slight flexibility of the deck members, and can be ignored. Participation factors are for X direction excitation. For Y direction, mode 2 has 100% participation. Details are summarized below.

NODE	COMP.	TRANSVERSE MODE 2 ϕ_i	LONGITUDINAL MODE 1 ϕ_i	MODE 3 ϕ_i
1	X	0	0.557	1.000
	Y	1.0	−0.747	0.747
3	X	0	0.557	1.000
	Y	1.0	+0.747	−0.747
5	X	0	1.000	0.557
	Y	1.0	−0.747	0.747
7	X	0	1.000	0.557
	Y	1.0	+0.747	0.747
PERIOD		0.602 sec	0.743 sec	0.503 sec
PARTICIPATION FACTOR		100%	64.2%	64.2%.

5.2.4 MODAL ANALYSIS

Ruaumoko is primarily a program for time-history analysis and pushover. It does not automatically perform modal combinations, so these are carried out by hand below.

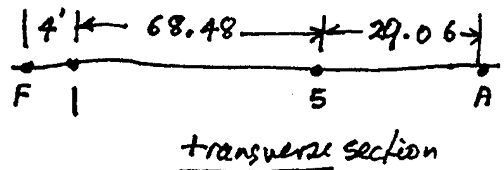
(a) Transverse Response

The response is identical to the Method A, with $T = 0.602$ sec., and 100% participation (see p C22)

$$\therefore \Delta_T = \underline{3.78 \text{ in}}$$

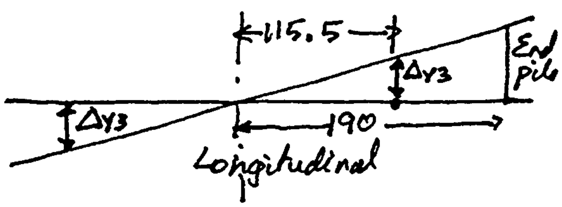
(b) Longitudinal Response

The mode shapes on p C27 refer to displacements at the "superpile" locations, and need to be extrapolated to the corner piles for maximum response:



$$\Delta_{XF} = \Delta_{X1} + \frac{(\Delta_{X1} - \Delta_{X5})}{68.48} \times 4$$

$$\Delta_{XA} = \Delta_{X5} - \frac{(\Delta_{X1} - \Delta_{X5})}{68.48} \times 29.06.$$



$$\Delta_{Yend} = \Delta_{Y3} \times \frac{190}{115.5} = 1.645 \Delta_{Y3}$$

Response in each of Modes 1 & 3 is given by

$$\underline{\Delta_i = p_i \phi_i S_{\Delta i}}$$

p_i = participation factor = 0.642
$S_{\Delta i}$ = Spectral acc, mode i.

Mode 1: $T = 0.743$ sec. From Table 1, $S_a = 0.941g$; $S_{\Delta 1} = \frac{T^2}{4\pi^2} S_a = \frac{0.743^2}{4\pi^2} \times 0.941 \times 386.2$

$S_{\Delta 1} = \underline{5.082 \text{ in}}$

Mode 3: $T = 0.503$s. $S_a = 1.147g$. $S_{\Delta 3} = \frac{0.503^2}{4\pi^2} \times 1.147 \times 386.2 = \underline{2.843 \text{ in.}}$

Node	Cpt	Mode 1 ϕ_i	Mode 1 Δ_i (in)	Mode 3 ϕ_i	Mode 3 Δ_i (in)	$\sqrt{M_1^2+M_3^2}$ $\sqrt{\Delta_1^2+\Delta_3^2}$ (in)
Row F, End	X	0.531	1.732	1.026	1.873	2.551
	Y	-1.229	-4.01	1.229	2.243	4.595
Row A, End	X	1.188	3.876	0.369	0.674	3.934
	Y	1.229	4.01	-1.229	-2.243	4.594

Note: ϕ_i from pC26 modified for end nodes as per pC28

Combine X & Y excitation Refer Eqns 3-17, 3-18 p3-45

100% X + 30% Y excitation:

Pile F: $\Delta_{XX} = 2.551$ $\Delta_{XY} = 0.3 \times 0.0$ $\therefore \Delta_X = 2.551$ in

$\Delta_{YX} = 4.595$ $\Delta_{YY} = 0.3 \times 3.78 = 1.134$ $\therefore \Delta_Y = 5.729$ in.

$\therefore \Delta_{max} = \sqrt{2.551^2 + 5.729^2} = \underline{6.271 \text{ in.}}$

Pile A: $\Delta_{XX} = 3.934$ $\Delta_{XY} = 0.3 \times 0.0 \rightarrow \Delta_X = 3.934$ in

$\Delta_{YX} = 4.594$ $\Delta_{YY} = 1.134$ $\therefore \Delta_y = 5.729$ in

$\therefore \Delta_{max} = \sqrt{3.934^2 + 5.729^2} = \underline{6.95 \text{ in.}}$

30% X + 100% Y excitation.

Pile F: $\Delta_{XX} = 0.3 \times 2.551$ $\Delta_{XY} = 0$ $\therefore \Delta_X = 0.765$

$\Delta_{YX} = 0.3 \times 4.595$ $\Delta_{YY} = 3.78$ $\therefore \Delta_Y = \underline{5.16 \text{ in}}$

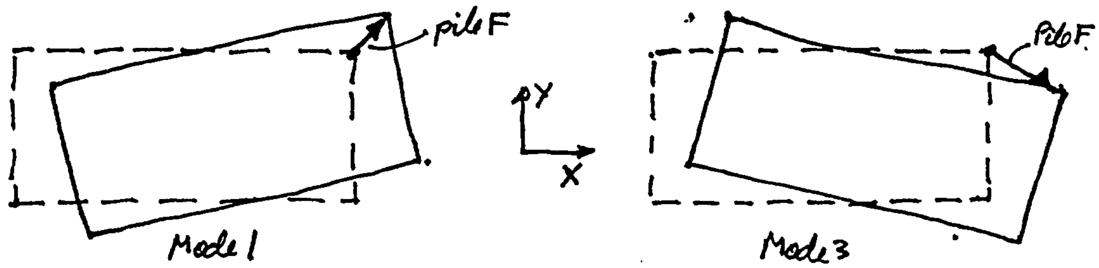
$\therefore \Delta_{max} = \sqrt{0.765^2 + 5.16^2} = \underline{5.22 \text{ in.}}$

PILE A: $\Delta_{XX} = 0.3 \times 3.934$ $\Delta_{XY} = 0$ $\therefore \Delta_X = 1.18$ in

$\Delta_{YX} = 0.3 \times 4.594$ $\Delta_{YY} = 3.78$ $\therefore \Delta_Y = 5.16$ in

$\therefore \Delta_{max} = \sqrt{1.18^2 + 5.16^2} = \underline{5.29 \text{ in.}}$

Note that the method B calculations result in maximum displacements for the F line pile about 18% higher than the value of 5.32in. predicted by Method A [p C23]. Close examination indicates this is due to a fundamental error in the modal combination approach, not in Eqn 3-19. Consider the deformation shapes of Modes 1 + 3:



Note the displacement vectors are orthogonal. Thus if we use SRSS or CQC to add the X component of displacement, the Y displacements should subtract: i.e, the combination for the Y components should be $\sqrt{\Delta_{Y_1}^2 - \Delta_{Y_2}^2}$. The Table at top of p C28 thus becomes:

Node	Cpt	Mode 1 ϕ_i	Mode 1 Δ_i (in)	Mode 3 ϕ_i	Mode 3 Δ_i	SRSS $\sqrt{\Delta_1^2 [\pm] \Delta_2^2}$	(− for Y cpts).
Row F End	X	0.531	1.732	1.026	1.873	2.551	$(\sqrt{1.732^2+1.873^2})$
	Y	-1.229	-4.01	1.229	2.243	3.324	$(\sqrt{4.01^2-2.243^2})$
Row	X	1.188	3.876	0.369	0.674	3.934	$(\sqrt{3.876^2+0.674^2})$
Row A End	Y	1.229	4.01	-1.229	-2.243	3.324	$(\sqrt{4.01^2-2.243^2})$

<u>Critical pile F displacements:</u>

100%X+30%Y: $\Delta_x = 2.551$; $\Delta_y = 3.324 + 0.3 \times 3.78 = 4.458$

$\therefore \Delta_{max} = \sqrt{2.551^2 + 4.458^2} = \underline{5.14 in}$ cf <u>5.32, method A.</u>

30%X+100%Y $\Delta_x = 0.765$, $\Delta_y = 0.3 \times 3.324 + 3.78 = 4.778$

$\therefore \Delta_{max} = \sqrt{0.765^2 + 4.778^2} = \underline{4.84 in}$

5.3 METHOD C ANALYSIS

5.3.1 Pure Transverse Analysis

Method C uses the results of the inelastic pushover analysis (Fig 15) and the substitute structure approach (effective stiffness + damping at maximum response) to estimate transverse displacement demand. Refer pp 3-54 to 3-59.

Fig 15 and Table 2 provide the backbone for the hysteretic response. The method of analysis requires the following steps

1. Guess the maximum displacement
2. Calculate the effective stiffness at max. displacement
3. Calculate effective period at max. displacement
4. Calculate effective damping at max. displacement
5. From response spectrum, calculate maximum displacement
6. Iterate 1 → 5 till convergence.

Based on the Method A analysis, an initial estimate of $\Delta_t = 4.0$ in. is made.

• Effective stiffness: From Table 2: $V_{4.0} = 436.3$ kips / 20 ft.

$$\therefore K = \frac{436.3}{4.0} = \underline{109.1 \text{ kip/in.}}$$

· Effective period:

$$T = 2\pi\sqrt{\frac{M}{K}}$$

$$= 2\pi\sqrt{\frac{1022.3}{386 \times 109.1}}$$

$$= \underline{0.978 \text{ sec}}.$$

: Effective damping: Assuming a Takeda hysteretic response, an explicit relationship between damping and ductility exists:

$$\xi = 0.05 + \frac{1}{\pi}\left(1 - \frac{(1-r)}{\sqrt{\mu}} - r\sqrt{\mu}\right)$$

μ = displacement ductility, r = second slope stiffness ratio.

From Fig 15, $r = 0.058$

$$\mu = \frac{\Delta_t}{\Delta_y} = \frac{4.0}{1.32} = 3.03 \quad (\Delta_y = 1.32 \text{ for bilinear approx. Fig 15})$$

$$\therefore \xi = 0.05 + \frac{1}{\pi}\left(1 - \frac{(1-0.058)}{\sqrt{3.03}} - 0.058\sqrt{3.03}\right)$$

$$= 0.164 \qquad (16.4\%).$$

<u>Maximum displacement</u>

The response spectrum is given for 5% only. The following relationship, from Eurocode 8 is used to reduce response for different damping levels:

$$R_\xi = \left(\frac{7}{2+\xi}\right)^{\frac{1}{2}} \qquad (\xi \text{ in } \%)$$

$$= \left(\frac{7}{18.4}\right)^{\frac{1}{2}}$$

$$= 0.617$$

For $T = 0.978$, from Table 1, $S_{a5} = 0.739g$

$\therefore$ 5% spectral displacement: $S_{\Delta 5} = \frac{T^2}{4\pi^2} \cdot g S_{a5}$

$$= \frac{0.978^2}{4\pi^2} \times 386.2 \times 0.739 \text{ in.}$$

$$= \underline{6.915 \text{ in.}}$$

$\therefore$ Correct for 16.4% damping $\quad \Delta = R_\xi \times S_{\Delta 5}$

$$= 0.617 \times 6.915$$

$$= \underline{4.27 \text{ in.}}$$

This is larger than the initial estimate, and iteration is required. To speed convergence, guess $\Delta = 4.40$ in.

<u>Effective stiffness</u> $\quad V_{4.4} = 444$ kips $\quad$ (Table 2)

$$\therefore K = \frac{444}{4.4}$$

$$= 100.9 \text{ kip/in.}$$

Effective Period: $T = 2\pi\sqrt{\frac{1022.3}{386 \times 100.9}} = 1.017\,sec.$

Effective Damping: $\mu = 4.40/1.32 = 3.33$

$$\therefore\ \xi = 0.05 + \frac{1}{\pi}\left(1 - \frac{0.942}{\sqrt{3.33}} - 0.058\sqrt{3.33}\right) = 0.1703$$

Max. Displacement: $R_\xi = \left(\frac{7}{19.03}\right)^{\frac{1}{2}} = 0.607$

For $T = 1.017$, from Table 1, $S_{a5} = 0.714$

5% spectral displacement $S_{\Delta 5} = \frac{1.017^2}{4\pi^2} \times 386.2 \times 0.714$

$= 7.224$ in.

17% damping disp. $\Delta = 0.607 \times 7.224$

$= 4.385$ in.

Say $\underline{\underline{\Delta_t = 4.39\text{ in.}}}$

5.3.2 Correction for combined longitudinal and transverse response.

Note: Eccentricity is different from Method A, due to reduction in stiffness at maximum response of piles F, E + D. Use Table 1, and forces at 4.5in: $V_F = 148.7$ kips, $V_E = 76.0$, $V_D = 36.6$, $V_C = 18.0$ $V_B = 10.4$, $V_A = 6.1$

Eccentricity from F line: $\bar{x}_F = \frac{(20 \times 76) + (40 \times 36.6) + (60 \times 18) + (80 \times 10.4) + (100 \times 6.1)}{444}$

$= \underline{12.4\text{ ft}}$

$\therefore$ Eccentricity between center of mass + center of stiffness is

$e = 55.44 - (12.4 + 3.5)$ refer PC1, C22.

$= 39.54$ ft.

From Eqn 3-19, amplification factor is

$$\sqrt{1+(0.3(1+20\times39.54/400))^2} = 1.34$$

$$\therefore \Delta_{max} = 4.39\times1.34$$

$$= \underline{\underline{5.88\,in.}} \quad (10.6\%\ \text{higher than Method A}).$$

Check Under Level 1 EQ. (40%)

Guess average displ = 1.6 in. $\longrightarrow \mu = 1.6/1.32 = 1.212$

From Table 2: $K = \dfrac{342.8}{1.6} = 214.3\,kip/in.$

$$T = 2\pi\sqrt{\frac{1022.3}{386\times214.3}} = 0.698\,sec.$$

$$S_{A5} = 0.978\,g.$$

Damping: $\xi = 0.05 + \frac{1}{\pi}\left(1 - \frac{0.942}{\sqrt{1.212}} - 0.058\sqrt{1.212}\right) = 7.56\%.$

$$R_\xi = \left(\frac{7}{9.56}\right)^{\frac{1}{2}} = 0.856$$

$$S_{\Delta5} = \frac{0.698^2}{4\pi^2}\times386\times0.4\times0.978 = 1.86\,in.$$

$$\therefore \Delta_{max} = 1.39\times(1.86\times0.856) \quad (1.39 = \text{factor for } e = 44\,ft)$$

$$= \underline{\underline{2.21\,in}}$$

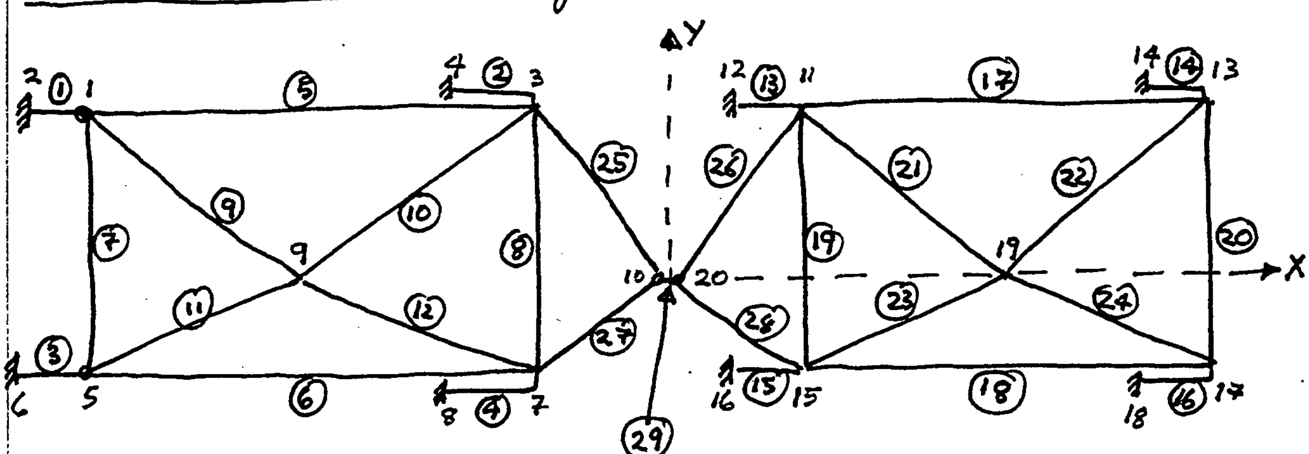
(4% larger than for Method A, and 1.5% above limit deflection).

5.4 METHOD D ANALYSIS - INELASTIC TIME HISTORY

In this section a full inelastic time-history analysis of the two-segment linked wharves is carried out. The data is similar to that for Method B, except two-wharf segments are used, linked by a pin at the common shear key. Each wharf segment is identical to the wharf segment for Method B, except that bilinear force-displacement characteristics are used for the "superpiles". Fig 18 shows the characteristics for a 20ft tributary length of deck. These need to be multiplied by 10 for the 200ft tributary length modeled by each "superpile". The backbone curve for each pile is described by the following data:

Type 1 Pile (2F+E+D)		Type 2 Pile [C+B+A]	
Δ (in)	F (kips)	Δ in	F kips
0.0	0.0	0.0	0.0
1.297	3630	4.0	326
5.00	4203	7.0	480.4

5.4.1. ANALYTICAL MODEL (Refer also to pC24, C25).



Nodes thus: 1 to 20 Members thus: (1) to (29)

Data [Input and Data Intepretation] are included in Appendix A.

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

Note: The coordinate system origin is at the center of the shear key member 29. The shear key member is rigidly connected to node 20 and pinned to node 10. However, under pure transverse response or pure longitudinal response there will be no relative rotation at node 10. This is obvious for transverse [Y] response. For longitudinal [X] response, considerations of antisymmetry indicate that the line through nodes 9, 10, 20, 19 must remain straight, and transverse displacement at the center of member 29 must be zero. This was confirmed by the dynamic analysis.

Note: The plan simulation and use of bi-axial springs to represent the superpiles means that the 3-D problem can be accurately simulated in 2-D. This is only possible because the deck is so stiff in the out-of-plane direction.

Note: Elastic damping taken as 2% [relative to initial stiffness].

5.4.2 EXCITATION

- Basic excitation was the NS component of the 1940 El Centro record, scaled x1.5, to give a PGA of 0.5g, and reasonably close spectral match with the design spectrum (Table 1) over the critical 0.5s to 1.0s range.
- First 20sec. applied transversely, then longitudinally.
- Final run carried out with both El Centro components simultaneously (NS in X and EW in Y).

5.4.3. RESULTS OF ANALYSIS

Results are plotted as time-histories of key nodal displacements, and shear key shear force, and are discussed in the

main report.

Check shear key force.

From Eqn 3-20, $V_{sk} = [1.5\, e/L_s]\, V_{\Delta T}$

For $\Delta_T = 3.8$ in (p C22) $V_{\Delta T} = 20 \times 436$ kip $=$ 8720 kips [Table 2]

$e = 46.06$ ft [p C22)

$\therefore$ Method A estimate is

$$V_{sk} = 1.5 \times \frac{46.06}{400} \times 8720$$

$$= \underline{1520 \text{ kips}}$$

From pure longitudinal excitation, $V_{sk} = 1800$ kips (Fig 22)

dual long + trans. " $V_{sk} = 1490$ kips.

Eqn 3-20 is 15% low for pure long. excitation
+ 2% high for dual excitation. } Reasonable.

6. SHEAR STRENGTH CHECK

Refer to pp3-76 to 3-79 of the resource document. The cases of piles confined by W20@2.5in. and W11@ 6in will be considered separately.

6.1 W20@2.5in.

Analyses in section 5 of these calculations indicated that the flexural displacement capacity of 8.55in. exceeded demand for all analytical methods. The maximum displacement demand was Δ_{max} = 5.88in. [Method C, pC32]. Use this to determine shear capacity.

6.1.1 Shear Demand

The maximum shear force in a pile occurs in F row piles. From Table 2, at $\Delta = 5.88$ in, V_D = 148.83 kips. This is the maximum shear force corresponding to design values for concrete, dowels, and foundation material. Higher shear forces would result from higher than specified concrete compression strength or dowel yield strength, or from stronger than estimated soil strength. Either carry out new pushover, using high estimates of structural and soil material strengths, or use default amplification factor of 1.4 (p3-76). We use the latter option

$$\therefore V_{max} = 1.4 \times 148.83$$

$$= \underline{208.4 \text{ kips}}$$

6.1.2. Concrete Mechanism Strength

$$V_c = k\sqrt{f'_c} \cdot A_e \qquad A_e = 0.8A_g = 0.8 \times 452 = \underline{361.7 in^2}$$

k depends on curvature ductility [Fig 3-30].

Calculate curvature ductility demand from $\Delta_{max} = 5.88$ in.

plastic displacement: $\Delta_p = \Delta_{max} - \Delta_y$ $\Delta_y = 1.03$ in (p C15)

$= 5.88 - 1.03 = \underline{4.85 \text{ in}}$

plastic rotation: $\theta_p = \Delta_p / L$ $L = 114''$ between hinges (p C15).

$= 4.85/114$

$= 0.0425$ radians.

plastic curvature: $\phi_p = \theta_p / L_p$ $L_p = 25.15''$ (p C16)

$= 0.0425/25.15$

$= 16.92 \times 10^{-6}/\text{in}$

curvature ductility: $\mu_\phi = \phi_{max}/\phi_y$ $\phi_y = 234 \times 10^{-6}/\text{in}$

$= (\phi_y + \phi_p)/\phi_y$

$= 1 + 1692/234$

$= \underline{8.2}$

Note: • Since the critical plastic hinge occurs at the pile top, all data refer to the dowel section, not the prestressed section.

• Since the pile could be subjected to ductility demand in orthogonal directions, use biaxial ductility curve in Fig 3-30.

• Use assessment curve rather than design curve.

From Fig 3-30: $k = 1.2 - \frac{3.2}{8} \times 0.6 = 0.96$ [psi units]

i.e. $V_c = 0.96\sqrt{f'_c} \times A_e$

$= 0.96\sqrt{7000} \times 361.7$ lbs ($f'_c = 7000$ conservative since shear amplified)

$= \underline{\underline{29.1 \text{ kips}}}$

6.1.3. Truss Mechanism

$V_s = \frac{\pi}{2} A_{sp} f_{yh} (D_p - c - c_o) \cot\theta / s$ (Eqn 3-37)

where $A_{sp} = 0.20\ in^2$,

$$\therefore V_s = \frac{\pi}{2} \times \frac{0.2 \times 70(24-7.3-3.25) \times 1.732}{2.5}$$

$f_{yh} = 70ksi$

$D_p = 24''$

$= \underline{204.9\ kips.}$

$c = N.A.depth = 7.3''$ [p. A-10]

$c_o = 3.0'' + \frac{0.5}{2} = 3.25''$

$\theta = 30°$ (assessment, not design)

$s = 2.5''$ (pitch)

6.1.4 Axial Force Mechanism

Strictly, the shear demand and shear strength should be checked for maximum and minimum values of Axial Force (remember axial force influences flexural strength and hence shear demand). However, influence on demand and strength is similar, and we check here only for $N_u = 100kips$.

$V_p = \phi[N_u + F_p] \tan\alpha$ (Eqn 3-39) $\phi = 1$ (assessment)

$F_p = 0$ [ignore prestress at pile top]

$= 100 \times [24 - (\frac{7.3+8.6}{2})]/114$ $\tan\alpha = (D - [c_{top} + c_{in.ground}]/2)/114$

7.3" (A-10) 8.6" (A-3)

$= \underline{14.1 kips}$

6.1.5 Nominal shear Strength

$V_N = V_c + V_s + V_p$

$= 29.1 + 204.9 + 14.1$

$= \underline{\underline{248.1\ kips}} > V_{max} = 208.3\ kips$ (pC36) <u>SHEAR OK</u>

6.2 W11 @ 6.0 in.

Flexural failure expected at $\Delta_{max} = 3.63\ in$ [pC16] $< 5.88\ in.$ demand. Check if shear failure preceeds flexural failure.

6.2.1 Shear Demand. Essentially unchanged (see Table 2 @ DIST = 3.6").

$\therefore V_{max} = \underline{208.4 \text{ kips}}$ (includes 40% overstrength).

6.2.1. Concrete Mechanism Strength.

Curvature limits can be used direct from pC16

At 3.63": $\mu_\phi = 1139/234$

$= \underline{4.9}$

From Fig 3-30: $k = 1.26$

$\therefore V_c = 1.26\sqrt{7000} \times 361.7$ lb.

$= \underline{38.1 \text{ Kips.}}$

6.2.2 Truss Mechanism Strength

$A_{sp} = 0.11 in^2$, $s = 6 in.$

$$V_s = \frac{\pi}{2} \times 0.11 \times 70 \left[\frac{24-7.3-3.25}{6}\right] \cot 30$$

$= \underline{47.0 \text{ kips}}$

6.2.3. Axial Force Mechanism

Essentially unchanged.

$V_P = \underline{14.1 \text{ kips}}$

6.2.4 Nominal shear strength @ 3.63 in.

$V_N = 38.1 + 47.0 + 14.1$

$= \underline{99.2 \text{ kips}} < 148.8 \ll 208.3$ SHEAR FAILURE CERTAIN

Thus shear failure will occur before maximum flexural displacement capacity is developed.

6.2.5 check shear at yield displacement

At $\Delta_y = 1.03 in$, $V_D = 118$ kips. $V_{max} = 1.4 \times 118 = 165.2$ (Table 2).

Concrete: $k = 3.5 \quad \therefore V_c = 3.5\sqrt{7000} \times 361.7$

$= 105.9$ kips.

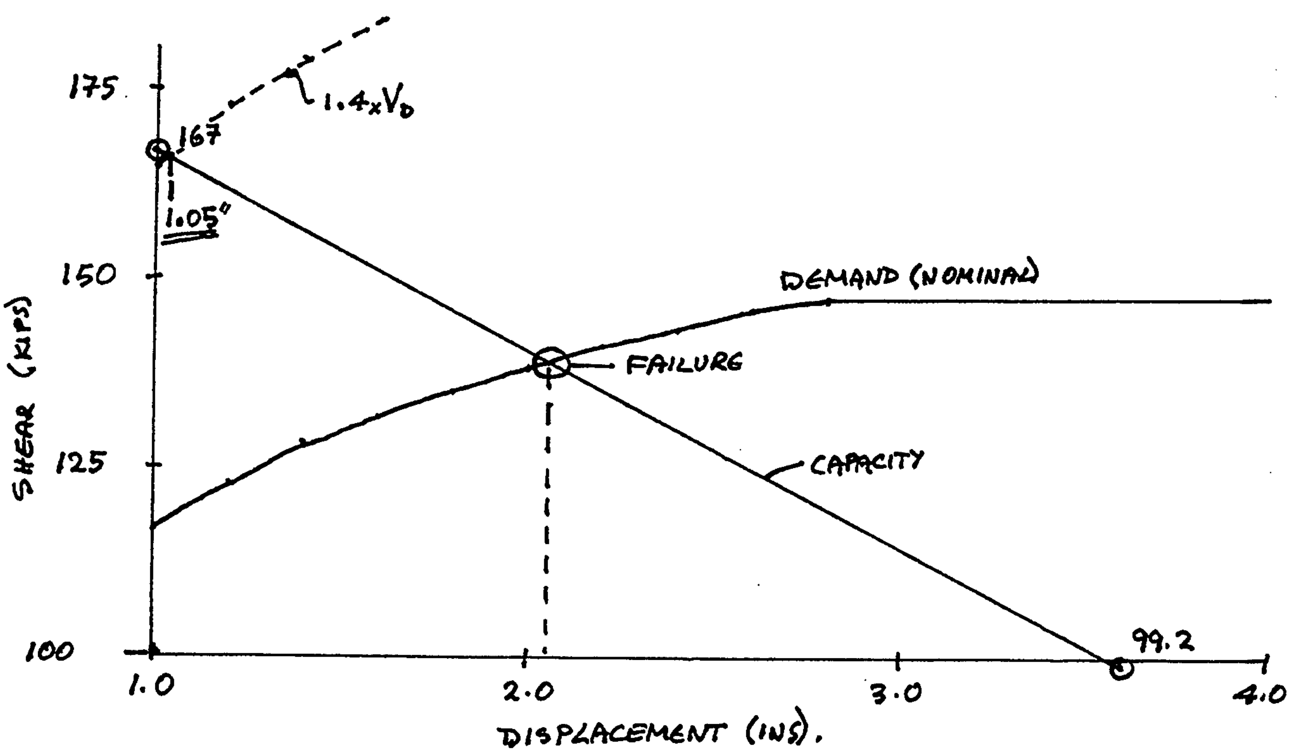
$V_s = 47.0$ kips (unchanged)

$V_p = 14.1$ kips (unchanged)

$\therefore \quad V_N = 105.9 + 47.0 + 14.1$

$= \underline{167.0 \text{ kips}}$

This is higher than $V_D = 118$ kips, but essentially equal to the amplified shear of 165.2 kips (p39). A conservative assessment would thus predict shear failure at $\Delta_{max} = \underline{1.03 \text{ in.}}$ A best-estimate can be found ignoring amplification, and plotting shear strength and shear demand against displacement - see below.



COMPARISON OF SHEAR DEMAND + CAPACITY. W11@6"

Thus: based on nominal demand, shear failure is expected at 2.05"

based on maximum feasible " " " " " 1.05"

Flexural failure expected at 3.63"

7. CHECK OF CONNECTION STRENGTH

7.1 W20 @ 2.5 in.

The well-detailed option of Fig 3a has the same confinement detail in the joint as in the pile (W20 @ 2.5 in). Anchorage and joint shear need to be checked.

7.1.1 Anchorage of Dowels in Deck.

Dowels are anchored as straight bars 3" from top of deck. Thus anchorage length = 30 in. (33-3).

Required anchorage length: $l_e = 0.025 d_b \frac{f_{ye}}{\sqrt{f'_c}}$ (Eqn 3-41b)

$$= 0.025 \times 1.27 \times \frac{66{,}000}{\sqrt{7000}}$$

$$= \underline{25.05'' < 30''} \quad \text{OK}$$

Anchorage is adequate.

7.1.2. Confinement for Joint shear Strength

Provided: W20 @ 2.5": $\rho_{sp} = \frac{4A_{sp}}{D's}$ $\quad D' = 24 - 2\times3 - \frac{1}{2}" = 17.5"$, $s = 2.5"$

$$= \frac{4 \times 0.2}{17.5 \times 2.5}$$

$$\therefore \underline{\rho_{sp} = 0.0183}$$

Required: $\rho_{sr} = \frac{0.46 A_{sc}}{D' l_a}\left[\frac{f^o_{yc}}{f_s}\right]$

$f^o_{yc} = 1.4 \times 60$ ksi
$f_s = 0.0015 E_s = 43.5$ ksi
$A_{sc} = 8\#10 = 10.16\, in^2$
$l_a = 30''$ [see above)

$$= \frac{0.46 \times 10.16}{17.5 \times 30}\left[\frac{1.4\times60}{43.5}\right]$$

$$\rho_{sr} = \underline{0.0172} < \rho_{sp}$$

$\therefore$ Joint reinforcement is adequate.

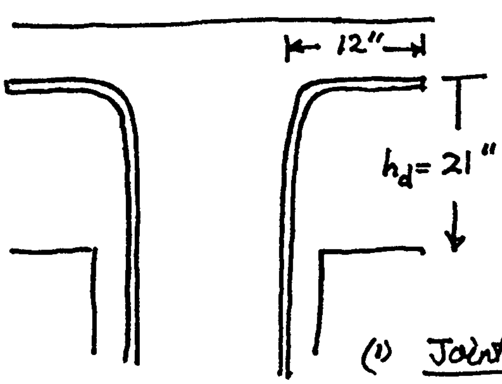
7.2 Detail in Fig 36: Joint has bars bent out 12" below deck top, and no joint shear reinforcement.

7.2.1 Anchorage

Total anchorage length, including hook, exceeds 25.05 in [see p C41]

∴ anchorage is OK.

7.2.2. Joint shear. — No joint reinforcement.



Nominal moment capacity $= M_N = 5692$ kip in (p C9)

Max. moment cap. $1.4 M_N = 7970$ kip in $= M^o$

(Alternatively, recalculate M^o using $1.7 f'_c$ and $1.3 f_y$ — a lower value of M^o will result.

(1) Joint shear: $$v_j = \frac{M^o}{\sqrt{2}\, h_d D_p^2} \qquad (\text{Eqn } 3\text{-}43)$$

$$= \frac{7970 \times 10^3}{\sqrt{2} \times 21 \times 24^2} \text{ psi}$$

$$= 466 \text{ psi}$$

(2) Axial stress for $N_u = 100$ kips: $$f_a = \frac{-100 \times 10^3}{(24+21)^2} \qquad (\text{Eqn } 3\text{-}45)$$

$$= -49.4 \text{ psi}$$

(3) principal tension stress: $$p_t = -24.7 + \sqrt{24.7^2 + 466^2}$$

$$= 442 \text{ psi} = 5.28\sqrt{f'_c} \qquad (f'_c = 7000)$$

$p_t = 5.28\sqrt{f'_c}$ exceeds $5.0\sqrt{f'_c}$ (marginally). Thus a joint shear failure is expected at less than the flexural overstrength moment capacity (but not at the nominal capacity).

It is probable that pile shear failure will preceed joint shear failure, and it is thus unnecessary to check the degrading moment capacity of the connection. However, the procedure is carried out below as an example.

7.2.3 Moment capacity corresponding to $p_t = 5\sqrt{f'_c}$ psi = ~~4..~~ 418psi.

$$v_j = \sqrt{p_t(p_t - f_a)} \qquad \text{Eqn 3-46}$$

$$= \sqrt{418(418+49.4)}$$

$$= \underline{442\,\text{psi.}}$$

$$M_C = \sqrt{2}\, v_j h_d D_p^2 \qquad \text{Eqn 3-47}$$

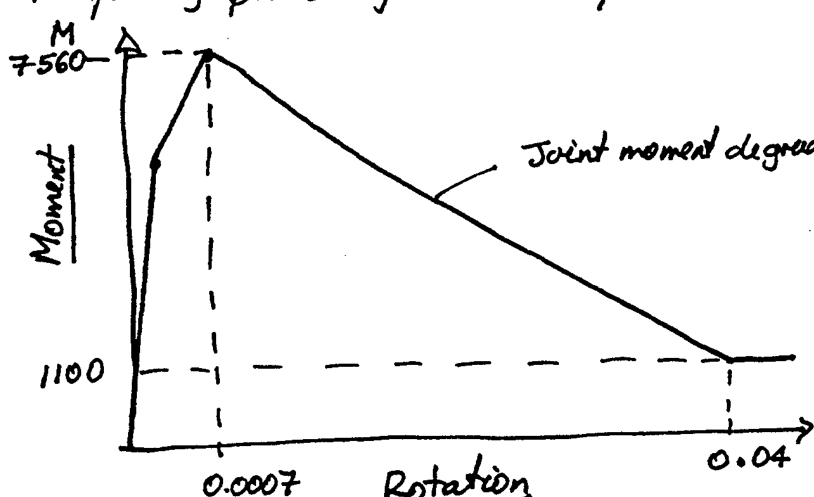
$$= \sqrt{2} \times 0.442 \times 21 \times 24^2 \text{ kip in}$$

$$= \underline{7560\,\text{kip in.}} < M^o \text{ but } > M_N.$$

The joint tension strength will degrade as the joint rotates. At a joint rotation of 0.04, $p_t = 0$, and the residual moment capacity is provided solely by the axial load of 100kips.

Thus $M_r \approx 100\left[\frac{24}{2} - 1\right] \approx \underline{\underline{1100K''}}$

The falling branch joint moment/rotation relationship is thus



Note: If the deck has stirrups surrounding the pile, moment capacity will be increased. Refer "Seismic Design of Bridges" by Priestley Seible, Calvi [John Wiley] p 565-569

SEISMIC ASSESSMENT OF A TWO-SEGMENT MARGINAL WHARF

COMPUTER INPUT/OUTPUT APPENDIX

Note:

- Data listed on pA1 for the moment-curvature analyses include tendon locations defined by distance from the tensile edge of the section. Since there are two tendons at each of the 8 distances (see Fig.2, e.g.) the tendon areas given are for two tendons at each location.
- The analysis program expects some mild steel in the section. A nominal minimal value of $0.01 in^2$ has been assumed. This will have negligible influence on the computed moment-curvature response.
- Refer to Fig.2 for section dimensions of the typical pile.
- Analyses are carried out for piles with W20 (area =0.2 in^2) wire spirals at 2.5", and W11 (area = 0.11 in^2) wire spirals at 6". In each case, three levels of axial load are considered: 0 (actually 0.5 kips, sine the program has problems with convergence criteria when zero axial load is specified), 100 kips, and 200 kips. This range exceeds the range of expected axial loads. For intermediate load values, data are found by interpolation

NAVDAT1

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FILE NAME : NAVDAT1

DATA INPUT
PRESTRESSED PILE
W20 @ 2.5"
MOMENT-CURVATURE
2 pages.

I N P U T D A T A(kips-in-unit)

DIAMETER OF SECTION	(in) :	24.00	
COVER TO MAIN STEEL	(in) :	3.50	
STEEL STRENGTH	(ksi) :	66.00	(high strength steel)
YOUNG'S MODULUS	(ksi) :	29000	
CONCRETE STRENGTH	(ksi) :	7.00	
APPLIED MOMENT	(kipsin) :	0	
APPLIED AXIAL LOAD	(kips) :	100.0	
MAIN BAR DIAMETER	(in) :	1.00	
TOTAL STEEL AREA	(sq in) :	0.10	
TIE DIAMETER	(in) :	0.51	
TIE SPACING	(in) :	2.50	
TIE STRENGTH	(ksi) :	70.00	(spirals)
NUMBER OF SEGMENTS	:	30	
ITERATION ACCURACY	:	0.0010	

THE PRESTRESSED TENDON NUMBER IS : 8

Point	y(in)	area(sqin)
1	20.042	0.43
2	18.818	0.43
3	16.556	0.43
4	13.600	0.43
5	10.400	0.43
6	7.444	0.43
7	5.182	0.43
8	3.958	0.43

Prestressed Steel Material Properties

1)	Initial steel Stress	(fsi)	(ksi)	:	154
2)	Yield stress	(fpy)	(ksi)	:	216
3)	young's modulus	(Es)	(ksi)	:	29000
4)	Ultimate steel strength	(fpu)	(ksi)	:	270
5)	Ultimate steel strain	(Epu)		:	0.06

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FILE NAME : p3dat

PRESTRESSED PILE, P=100kips, W20@2.5"

M O M E N T C U R V A T U R E A N A L Y S I S R E S U L T S(kips-in-unit)

CYCLE No.	CONC. STRAIN	N.A. DEPTH (in)	STEEL STRAIN	TENDON STRESS (ksi)	TENDON STRAIN	M. (Kips-in)	CURVATURE (1/in)
1	0.001000	15.12	-0.000323	-171.58	-.00591	3962.232	0.000066
2	0.001100	14.17	-0.000452	-175.35	-.00604	4243.606	0.000078
3	0.001200	13.40	-0.000591	-179.39	-.00618	4493.671	0.000090
4	0.001300	12.75	-0.000739	-183.68	-.00633	4719.226	0.000102
5	0.001400	12.21	-0.000892	-188.16	-.00648	4925.115	0.000115
6	0.001500	11.76	-0.001052	-192.80	-.00664	5115.077	0.000128
7	0.001600	11.37	-0.001215	-197.56	-.00681	5290.410	0.000141
8	0.001800	10.75	-0.001550	-207.31	-.00714	5607.202	0.000168
9	0.002000	10.35	-0.001866	-231.48	-.00746	5952.772	0.000193
10	0.002500	9.58	-0.002722	-232.36	-.00832	6374.290	0.000261
11	0.003000	9.15	-0.003554	-233.22	-.00915	6538.893	0.000328
12	0.003500	8.92	-0.004347	-234.04	-.00995	6548.993	0.000392
13	0.004000	8.84	-0.005047	-234.76	-.01065	6523.485	0.000452
14	0.004500	8.79	-0.005742	-235.48	-.01135	6434.651	0.000512
15	0.005000	8.77	-0.006397	-236.15	-.01201	6334.542	0.000570
16	0.006000	8.82	-0.007601	-237.40	-.01322	6112.454	0.000680
17	0.007000	8.92	-0.008697	-238.53	-.01432	5878.963	0.000785
18	0.008000	8.98	-0.009817	-239.68	-.01544	5717.336	0.000891
19	0.009000	9.00	-0.011007	-240.91	-.01664	5634.444	0.001000
20	0.010000	8.96	-0.012331	-242.28	-.01796	5631.286	0.001117
21	0.012000	8.90	-0.014955	-244.99	-.02060	5671.684	0.001348
22	0.014000	8.74	-0.018036	-248.17	-.02369	5771.917	0.001602
23	0.016000	8.61	-0.021152	-251.38	-.02682	5869.569	0.001858
24	0.018000	8.49	-0.024395	-254.73	-.03007	5976.632	0.002120
25	0.020000	8.39	-0.027676	-258.11	-.03336	6078.866	0.002384
26	0.022000	8.29	-0.031085	-261.63	-.03678	6184.981	0.002654
27	0.024000	8.20	-0.034563	-265.22	-.04027	6289.495	0.002928
28	0.026000	8.13	-0.037975	-268.74	-.04370	6379.204	0.003199
29	0.028000	8.10	-0.041128	-271.99	-.04686	6435.690	0.003456

CONCRETE STRAIN EXCEEDS MAXIMUM

CALCULATED IDEAL MOMENT 6549 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.02638
80% OF MAXIMUM MOMENT : 5239.2 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL : YES

APPLIED AXIAL LOAD : 100.0 (kips)

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FILE NAME : p3dat PRESTRESSED PILE, P=0.5kips, W20@2.5"

M O M E N T C U R V A T U R E A N A L Y S I S R E S U L T S(kips-in-unit)

CYCLE No.	CONC. STRAIN	N.A. DEPTH (in)	STEEL STRAIN	TENDON STRESS (ksi)	TENDON STRAIN	M. (Kips-in)	CURVATURE (1/in)
1	0.001000	13.37	-0.000496	-175.35	-.00604	3801.637	0.000075
2	0.001100	12.58	-0.000649	-179.78	-.00620	4038.789	0.000087
3	0.001200	11.94	-0.000810	-184.50	-.00636	4255.102	0.000101
4	0.001300	11.40	-0.000980	-189.44	-.00653	4455.188	0.000114
5	0.001400	10.95	-0.001156	-194.56	-.00670	4642.206	0.000128
6	0.001500	10.58	-0.001337	-199.82	-.00689	4817.933	0.000142
7	0.001600	10.25	-0.001521	-205.18	-.00707	4983.860	0.000156
8	0.001800	9.75	-0.001894	-216.04	-.00744	5289.355	0.000185
9	0.002000	9.45	-0.002235	-231.81	-.00779	5646.945	0.000212
10	0.002500	8.75	-0.003214	-232.82	-.00877	5983.429	0.000286
11	0.003000	8.35	-0.004185	-233.83	-.00974	6089.422	0.000359
12	0.003500	8.18	-0.005055	-234.72	-.01062	6125.073	0.000428
13	0.004000	8.09	-0.005893	-235.59	-.01146	6073.427	0.000495
14	0.004500	8.06	-0.006666	-236.38	-.01223	6000.949	0.000558
15	0.005000	8.07	-0.007387	-237.13	-.01296	5916.280	0.000619
16	0.006000	8.16	-0.008702	-238.49	-.01428	5722.998	0.000735
17	0.007000	8.29	-0.009886	-239.71	-.01546	5514.483	0.000844
18	0.008000	8.40	-0.011049	-240.91	-.01663	5409.933	0.000952
19	0.009000	8.44	-0.012327	-242.23	-.01791	5306.780	0.001066
20	0.010000	8.42	-0.013753	-243.70	-.01935	5281.735	0.001188
21	0.012000	8.35	-0.016731	-246.77	-.02233	5294.421	0.001437
22	0.014000	8.22	-0.020056	-250.20	-.02567	5381.572	0.001703
23	0.016000	8.11	-0.023462	-253.72	-.02909	5475.551	0.001973
24	0.018000	8.02	-0.026912	-257.28	-.03255	5569.944	0.002246
25	0.020000	7.92	-0.030475	-260.95	-.03612	5672.792	0.002524
26	0.022000	7.85	-0.034070	-264.66	-.03973	5773.102	0.002804
27	0.024000	7.78	-0.037696	-268.40	-.04337	5871.611	0.003085

PRESTRESS TENDON STRAIN EXCEED MAXIMUM

CALCULATED IDEAL MOMENT 6125 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.02638
80% OF MAXIMUM MOMENT : 4900.1 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL : YES

APPLIED AXIAL LOAD : 0.5 (kips)

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FILE NAME : p3dat PRESTRESSED PILE, P= 200KIPS, W20@2.5"

M O M E N T   C U R V A T U R E   A N A L Y S I S   R E S U L T S(kips-in-unit)

CYCLE No.	CONC. STRAIN	N.A. DEPTH (in)	STEEL STRAIN	TENDON STRESS (ksi)	TENDON STRAIN	M. (Kips-in)	CURVATURE (1/in)
1	0.001000	16.90	-0.000183	-168.79	-.00582	4020.751	0.000059
2	0.001100	15.79	-0.000293	-171.99	-.00593	4360.157	0.000070
3	0.001200	14.88	-0.000413	-175.47	-.00605	4654.633	0.000081
4	0.001300	14.13	-0.000540	-179.18	-.00617	4915.086	0.000092
5	0.001400	13.50	-0.000675	-183.10	-.00631	5148.299	0.000104
6	0.001500	12.96	-0.000815	-187.19	-.00645	5359.383	0.000116
7	0.001600	12.50	-0.000960	-191.41	-.00660	5551.418	0.000128
8	0.001800	11.77	-0.001259	-200.12	-.00690	5887.797	0.000153
9	0.002000	11.23	-0.001564	-208.99	-.00720	6171.601	0.000178
10	0.002500	10.43	-0.002292	-231.96	-.00793	6748.780	0.000240
11	0.003000	9.96	-0.003022	-232.72	-.00866	6943.561	0.000301
12	0.003500	9.68	-0.003733	-233.45	-.00938	6940.851	0.000362
13	0.004000	9.54	-0.004387	-234.12	-.01003	6881.358	0.000419
14	0.004500	9.51	-0.004962	-234.72	-.01061	6822.879	0.000473
15	0.005000	9.47	-0.005556	-235.33	-.01121	6709.243	0.000528
16	0.006000	9.48	-0.006656	-236.47	-.01231	6460.772	0.000633
17	0.007000	9.55	-0.007666	-237.51	-.01333	6204.286	0.000733
18	0.008000	9.58	-0.008702	-238.58	-.01437	6032.077	0.000835
19	0.009000	9.57	-0.009800	-239.71	-.01547	5940.588	0.000940
20	0.010000	9.51	-0.011037	-240.99	-.01671	5944.833	0.001052
21	0.012000	9.38	-0.013592	-243.63	-.01928	5994.021	0.001280
22	0.014000	9.24	-0.016313	-246.44	-.02201	6105.762	0.001516
23	0.016000	9.12	-0.019103	-249.31	-.02481	6233.654	0.001755
24	0.018000	8.98	-0.022078	-252.38	-.02779	6343.150	0.002004
25	0.020000	8.85	-0.025190	-255.60	-.03092	6457.902	0.002260
26	0.022000	8.74	-0.028356	-258.86	-.03409	6568.154	0.002518
27	0.024000	8.64	-0.031552	-262.16	-.03730	6677.872	0.002778
28	0.026000	8.60	-0.034498	-265.20	-.04026	6739.610	0.003025
29	0.028000	8.55	-0.037467	-268.26	-.04324	6800.627	0.003273

CONCRETE STRAIN EXCEEDS MAXIMUM

CALCULATED IDEAL MOMENT 6944 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.02638
80% OF MAXIMUM MOMENT : 5554.8 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL : YES

APPLIED AXIAL LOAD : 200.0 (kips)

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FILE NAME : N1 PRESTRESSED PILE, P=100kips, W11 @ 6"

MOMENT CURVATURE ANALYSIS RESULTS(kips-in-unit)

CYCLE No.	CONC. STRAIN	N.A. DEPTH (in)	STEEL STRAIN	TENDON STRESS (ksi)	TENDON STRAIN	M. (Kips-in)	CURVATURE (1/in)
1	0.001000	15.00	-0.000333	-171.88	-.00592	3988.687	0.000067
2	0.001100	14.07	-0.000464	-175.69	-.00605	4268.903	0.000078
3	0.001200	13.30	-0.000605	-179.78	-.00620	4518.150	0.000090
4	0.001300	12.66	-0.000753	-184.11	-.00634	4743.077	0.000103
5	0.001400	12.13	-0.000909	-188.64	-.00650	4948.569	0.000115
6	0.001500	11.68	-0.001070	-193.32	-.00666	5138.104	0.000128
7	0.001600	11.29	-0.001234	-198.12	-.00683	5313.417	0.000142
8	0.001800	10.68	-0.001572	-207.94	-.00717	5630.041	0.000169
9	0.002000	10.28	-0.001891	-231.50	-.00749	5972.608	0.000195
10	0.002500	9.52	-0.002751	-232.39	-.00835	6388.429	0.000263
11	0.003000	9.11	-0.003586	-233.25	-.00919	6547.490	0.000329
12	0.003500	8.89	-0.004370	-234.06	-.00997	6549.786	0.000394
13	0.004000	8.83	-0.005055	-234.77	-.01066	6514.478	0.000453
14	0.004500	8.80	-0.005722	-235.46	-.01133	6412.066	0.000511
15	0.005000	8.82	-0.006334	-236.09	-.01194	6294.172	0.000567
16	0.006000	8.95	-0.007411	-237.20	-.01303	6023.145	0.000671
17	0.007000	9.14	-0.008319	-238.14	-.01394	5722.381	0.000766
18	0.008000	9.30	-0.009197	-239.05	-.01482	5484.261	0.000860
19	0.009000	9.43	-0.010098	-239.98	-.01572	5321.291	0.000955
20	0.010000	9.46	-0.011137	-241.05	-.01677	5262.970	0.001057
21	0.012000	9.51	-0.013239	-243.22	-.01888	5205.831	0.001262

MOMENT LESS THAN 80% OF MAXIMUM MOMENT

CALCULATED IDEAL MOMENT 6550 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.01113
80% OF MAXIMUM MOMENT : 5239.8 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL : YES

APPLIED AXIAL LOAD : 100.0 (kips)

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CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER
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FILE NAME : N1 PRESTRESSED PILE, W11 @ 6", P=0.5 kips

M O M E N T C U R V A T U R E A N A L Y S I S R E S U L T S(kips-in-unit)

CYCLE No.	CONC. STRAIN	N.A. DEPTH (in)	STEEL STRAIN	TENDON STRESS (ksi)	TENDON STRAIN	M. (Kips-in)	CURVATURE (1/in)
1	0.001000	13.28	-0.000506	-175.63	-.00605	3819.927	0.000075
2	0.001100	12.50	-0.000660	-180.11	-.00621	4056.698	0.000088
3	0.001200	11.86	-0.000823	-184.86	-.00637	4272.801	0.000101
4	0.001300	11.33	-0.000994	-189.84	-.00654	4472.752	0.000115
5	0.001400	10.89	-0.001171	-195.00	-.00672	4659.840	0.000129
6	0.001500	10.51	-0.001353	-200.30	-.00690	4835.761	0.000143
7	0.001600	10.20	-0.001539	-205.70	-.00709	5001.859	0.000157
8	0.001800	9.74	-0.001898	-231.47	-.00745	5399.689	0.000185
9	0.002000	9.39	-0.002259	-231.84	-.00781	5660.275	0.000213
10	0.002500	8.70	-0.003245	-232.85	-.00880	5993.485	0.000287
11	0.003000	8.32	-0.004214	-233.85	-.00977	6094.247	0.000361
12	0.003500	8.16	-0.005083	-234.75	-.01064	6127.998	0.000429
13	0.004000	8.07	-0.005908	-235.60	-.01147	6070.311	0.000495
14	0.004500	8.07	-0.006658	-236.38	-.01222	5988.771	0.000558
15	0.005000	8.10	-0.007343	-237.08	-.01291	5891.595	0.000617
16	0.006000	8.25	-0.008544	-238.32	-.01412	5662.515	0.000727
17	0.007000	8.46	-0.009558	-239.37	-.01514	5402.531	0.000828
18	0.008000	8.63	-0.010531	-240.37	-.01611	5189.807	0.000927
19	0.009000	8.75	-0.011572	-241.45	-.01716	5063.196	0.001029
20	0.010000	8.82	-0.012687	-242.60	-.01828	4996.399	0.001134
21	0.012000	8.93	-0.014861	-244.85	-.02046	4944.998	0.001343

CONCRETE STRAIN EXCEEDS MAXIMUM

CALCULATED IDEAL MOMENT 6128 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.01113
80% OF MAXIMUM MOMENT : 4902.4 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL : YES

APPLIED AXIAL LOAD : 0.5 (kips)

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CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER
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FILE NAME : N1 PRESTRESSED PILE, P=200Kips, W11@6"

MOMENT CURVATURE ANALYSIS RESULTS(kips-in-unit)

CYCLE No.	CONC. STRAIN	N.A. DEPTH (in)	STEEL STRAIN	TENDON STRESS (ksi)	TENDON STRAIN	M. (Kips-in)	CURVATURE (1/in)
1	0.001000	16.76	-0.000194	-169.09	-.00583	4057.745	0.000060
2	0.001100	15.66	-0.000305	-172.34	-.00594	4395.093	0.000070
3	0.001200	14.76	-0.000426	-175.87	-.00606	4687.854	0.000081
4	0.001300	14.01	-0.000555	-179.63	-.00619	4946.945	0.000093
5	0.001400	13.39	-0.000692	-183.59	-.00633	5179.071	0.000105
6	0.001500	12.86	-0.000833	-187.73	-.00647	5389.280	0.000117
7	0.001600	12.40	-0.000980	-191.99	-.00662	5580.644	0.000129
8	0.001800	11.68	-0.001281	-200.77	-.00692	5915.833	0.000154
9	0.002000	11.15	-0.001588	-209.71	-.00723	6198.948	0.000179
10	0.002500	10.37	-0.002323	-231.99	-.00796	6766.138	0.000241
11	0.003000	9.91	-0.003052	-232.75	-.00870	6952.191	0.000303
12	0.003500	9.65	-0.003755	-233.47	-.00940	6940.919	0.000363
13	0.004000	9.54	-0.004389	-234.13	-.01004	6866.316	0.000419
14	0.004500	9.54	-0.004930	-234.69	-.01058	6786.229	0.000471
15	0.005000	9.55	-0.005476	-235.25	-.01113	6648.156	0.000524
16	0.006000	9.65	-0.006436	-236.24	-.01209	6334.684	0.000622
17	0.007000	9.83	-0.007244	-237.08	-.01290	5992.901	0.000712
18	0.008000	9.98	-0.008024	-237.88	-.01369	5725.362	0.000801
19	0.009000	10.09	-0.008840	-238.72	-.01451	5550.996	0.000892

MOMENT LESS THAN 80% OF MAXIMUM MOMENT

CALCULATED IDEAL MOMENT 6952 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.01113
80% OF MAXIMUM MOMENT : 5561.8 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL : YES

APPLIED AXIAL LOAD : 200.0 (kips)

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CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER
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FILE NAME : N2

I N P U T   D A T A(kips-in-unit)

DIAMETER OF SECTION	(in)	:	24.00
COVER TO MAIN STEEL	(in)	:	3.65
STEEL STRENGTH	(ksi)	:	66.00 (high strength steel)
YOUNG'S MODULUS	(ksi)	:	29000
CONCRETE STRENGTH	(ksi)	:	7.00
APPLIED MOMENT	(kipsin)	:	0
APPLIED AXIAL LOAD	(kips)	:	100.0
MAIN BAR DIAMETER	(in)	:	1.27
TOTAL STEEL AREA	(sq in)	:	10.16
TIE DIAMETER	(in)	:	0.51
TIE SPACING	(in)	:	2.50
TIE STRENGTH	(ksi)	:	70.00 (spirals)
NUMBER OF SEGMENTS		:	30
ITERATION ACCURACY		:	0.0010

DATA INPUT
DOWEL CONNECTION
W202 2.5"
MOMENT-CURVATURE

CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER
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FILE NAME : N2 DOWELL CONNECTION, P=100kips, W20@2.5"

M O M E N T   C U R V A T U R E   A N A L Y S I S   R E S U L T S(kips-in-unit)

CYCLE NUMBER	CONCRETE STRAIN	N.A. DEPTH (in)	STEEL STRAIN	MOMENT (kipsin)	CURVATURE (1/in)
1	0.000100	19.54	-0.000002	431.86	0.0000051
2	0.000200	12.34	-0.000122	833.62	0.0000162
3	0.000300	10.27	-0.000279	1161.12	0.0000292
4	0.000400	9.32	-0.000451	1485.16	0.0000429
5	0.000500	8.78	-0.000630	1810.90	0.0000569
6	0.000600	8.44	-0.000810	2136.21	0.0000711
7	0.000700	8.21	-0.000993	2461.88	0.0000853
8	0.000800	8.04	-0.001176	2786.98	0.0000996
9	0.000900	7.91	-0.001358	3110.31	0.0001138
10	0.001000	7.81	-0.001540	3430.67	0.0001280
11	0.001100	7.74	-0.001721	3747.91	0.0001422
12	0.001200	7.68	-0.001901	4060.81	0.0001563
13	0.001300	7.63	-0.002079	4368.78	0.0001703
14	0.001400	7.60	-0.002255	4671.14	0.0001842
15	0.001500	7.53	-0.002452	4906.40	0.0001991
16	0.001600	7.45	-0.002659	5104.36	0.0002146
17	0.001800	7.28	-0.003104	5398.45	0.0002472
18	0.002000	7.12	-0.003573	5608.03	0.0002808
19	0.002500	6.81	-0.004787	5915.40	0.0003673
20	0.003000	6.63	-0.005983	6043.88	0.0004527
21	0.003500	6.54	-0.007125	6077.89	0.0005355
22	0.004000	6.51	-0.008183	6086.90	0.0006140
23	0.004500	6.53	-0.009166	6088.90	0.0006887
24	0.005000	6.57	-0.010091	6077.52	0.0007605
25	0.006000	6.69	-0.011799	6027.99	0.0008970
26	0.007000	6.92	-0.013070	5883.64	0.0010115
27	0.008000	7.10	-0.014373	5787.57	0.0011275
28	0.009000	7.23	-0.015710	5729.69	0.0012453
29	0.010000	7.34	-0.017040	5691.33	0.0013627
30	0.012000	7.45	-0.019954	5722.45	0.0016104
31	0.014000	7.58	-0.022642	5737.79	0.0018466
32	0.016000	7.61	-0.025743	5829.85	0.0021037
33	0.018000	7.64	-0.028752	5911.59	0.0023562
34	0.020000	7.68	-0.031663	5976.22	0.0026037
35	0.022000	7.70	-0.034659	6044.88	0.0028555
36	0.024000	7.73	-0.037615	6105.27	0.0031052

NUMBER OF CYCLES EXCEEDS MAXIMUM OF 36

CALCULATED IDEAL MOMENT 6089 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.02778
80% OF MAXIMUM MOMENT : 4884.2 (kipsin)
STEEL LIMIT STRAIN : 0.120000

ITERATION ACCURACY :   0.001
STRAIN HARDENING OF STEEL

APPLIED AXIAL LOAD :   100.0 (kips)

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CIRCULAR COLUMNS - MANDER
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FILE NAME : N2 DOWELL CONNECTION, P=0.5 kips, W20∅2.5"

M O M E N T   C U R V A T U R E   A N A L Y S I S   R E S U L T S(kips-in-unit)

CYCLE NUMBER	CONCRETE STRAIN	N.A. DEPTH (in)	STEEL STRAIN	MOMENT (kipsin)	CURVATURE (1/in)
1	0.000100	6.91	-0.000187	334.67	0.0000145
2	0.000200	6.89	-0.000376	668.68	0.0000290
3	0.000300	6.89	-0.000564	1002.49	0.0000436
4	0.000400	6.89	-0.000753	1335.94	0.0000581
5	0.000500	6.89	-0.000941	1668.81	0.0000726
6	0.000600	6.89	-0.001129	2000.81	0.0000871
7	0.000700	6.89	-0.001316	2331.54	0.0001016
8	0.000800	6.89	-0.001503	2660.56	0.0001160
9	0.000900	6.90	-0.001688	2987.33	0.0001304
10	0.001000	6.91	-0.001873	3311.25	0.0001448
11	0.001100	6.91	-0.002057	3631.65	0.0001591
12	0.001200	6.93	-0.002238	3947.85	0.0001733
13	0.001300	6.90	-0.002441	4203.59	0.0001885
14	0.001400	6.84	-0.002661	4413.38	0.0002046
15	0.001500	6.77	-0.002895	4583.03	0.0002215
16	0.001600	6.70	-0.003139	4725.88	0.0002388
17	0.001800	6.55	-0.003654	4947.32	0.0002749
18	0.002000	6.41	-0.004190	5108.79	0.0003120
19	0.002500	6.15	-0.005568	5349.33	0.0004066
20	0.003000	6.00	-0.006917	5450.67	0.0004998
21	0.003500	5.95	-0.008177	5502.28	0.0005885
22	0.004000	5.95	-0.009335	5539.53	0.0006720
23	0.004500	5.99	-0.010408	5560.99	0.0007513
24	0.005000	6.04	-0.011420	5565.41	0.0008275
25	0.006000	6.18	-0.013273	5543.55	0.0009713
26	0.007000	6.37	-0.014804	5462.74	0.0010988
27	0.008000	6.57	-0.016175	5387.21	0.0012184
28	0.009000	6.72	-0.017569	5329.30	0.0013390
29	0.010000	6.82	-0.019091	5318.97	0.0014661
30	0.012000	6.98	-0.022102	5338.83	0.0017186
31	0.014000	7.09	-0.025205	5391.86	0.0019758
32	0.016000	7.18	-0.028225	5439.13	0.0022288
33	0.018000	7.20	-0.031580	5529.99	0.0024987
34	0.020000	7.23	-0.034895	5609.43	0.0027665
35	0.022000	7.26	-0.038137	5675.17	0.0030307
36	0.024000	7.29	-0.041317	5733.95	0.0032918

NUMBER OF CYCLES EXCEEDS MAXIMUM OF 36

CALCULATED IDEAL MOMENT 5565 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.02778
80% OF MAXIMUM MOMENT : 4587.2 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001

STRAIN HARDENING OF STEEL

APPLIED AXIAL LOAD : 0.5 (kips)

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CIRCULAR COLUMNS - MANDER

FILE NAME : N2 DOWEL CONNECTION, P=200kips, W20@2.5"

MOMENT CURVATURE ANALYSIS RESULTS(kips-in-unit)

CYCLE NUMBER	CONCRETE STRAIN	N.A. DEPTH (in)	STEEL STRAIN	MOMENT (kipsin)	CURVATURE (1/in)
1	0.000100	58.30	0.000066	154.10	0.0000017
2	0.000200	19.56	-0.000003	863.19	0.0000102
3	0.000300	14.62	-0.000107	1310.95	0.0000205
4	0.000400	12.37	-0.000242	1663.13	0.0000323
5	0.000500	11.10	-0.000394	1991.84	0.0000450
6	0.000600	10.30	-0.000556	2313.96	0.0000583
7	0.000700	9.75	-0.000724	2633.83	0.0000718
8	0.000800	9.36	-0.000896	2952.37	0.0000855
9	0.000900	9.06	-0.001071	3271.31	0.0000993
10	0.001000	8.84	-0.001245	3583.93	0.0001132
11	0.001100	8.66	-0.001421	3896.17	0.0001270
12	0.001200	8.52	-0.001596	4204.10	0.0001409
13	0.001300	8.40	-0.001769	4506.98	0.0001547
14	0.001400	8.31	-0.001942	4804.11	0.0001684
15	0.001500	8.24	-0.002112	5094.82	0.0001820
16	0.001600	8.18	-0.002281	5376.97	0.0001956
17	0.001800	8.00	-0.002662	5768.93	0.0002249
18	0.002000	7.82	-0.003073	6041.14	0.0002557
19	0.002500	7.46	-0.004146	6426.03	0.0003350
20	0.003000	7.25	-0.005215	6585.84	0.0004140
21	0.003500	7.14	-0.006233	6632.41	0.0004905
22	0.004000	7.09	-0.007199	6619.58	0.0005644
23	0.004500	7.08	-0.008106	6594.55	0.0006353
24	0.005000	7.11	-0.008956	6568.36	0.0007033
25	0.006000	7.25	-0.010424	6455.38	0.0008277
26	0.007000	7.45	-0.011655	6288.64	0.0009401
27	0.008000	7.62	-0.012832	6160.33	0.0010499
28	0.009000	7.73	-0.014088	6095.84	0.0011636
29	0.010000	7.85	-0.015273	6031.52	0.0012737
30	0.012000	7.94	-0.017991	6065.80	0.0015114
31	0.014000	8.03	-0.020611	6094.43	0.0017443
32	0.016000	8.05	-0.023437	6181.90	0.0019875
33	0.018000	8.08	-0.026183	6252.39	0.0022267
34	0.020000	8.09	-0.029024	6325.67	0.0024707
35	0.022000	8.12	-0.031757	6385.83	0.0027092
36	0.024000	8.15	-0.034404	6438.83	0.0029434

NUMBER OF CYCLES EXCEEDS MAXIMUM OF 36

CALCULATED IDEAL MOMENT 6632 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.02778
80% OF MAXIMUM MOMENT : 5305.9 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001

STRAIN HARDENING OF STEEL

APPLIED AXIAL LOAD :   200.0 (kips)

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CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER
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FILE NAME : N3

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 I N P U T   D A T A(kips-in-unit)

     DIAMETER OF SECTION     (in) :    24.00
     COVER TO MAIN STEEL     (in) :     3.65

         STEEL STRENGTH     (ksi) :    66.00 ( high strength steel )
        YOUNG'S MODULUS     (ksi) :    29000
      CONCRETE STRENGTH     (ksi) :     7.00

      APPLIED MOMENT(kipsin)      :        0
     APPLIED AXIAL LOAD    (kips) :    100.0

      MAIN BAR DIAMETER      (in) :     1.27
       TOTAL STEEL AREA  (sq in)  :    10.16

           TIE DIAMETER      (in) :     0.38
            TIE SPACING      (in) :     6.00
           TIE STRENGTH     (ksi) :    70.00 ( spirals )

     NUMBER OF SEGMENTS           :        30
     ITERATION ACCURACY           :          0.0010
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DATA INPUT
DOWELL CONNECTION
W11 @ 6 in
MOMENT - CURVATURE

CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER

FILE NAME : N3 DOWELL CONNECTION, P=100 kips, W11@6"

MOMENT CURVATURE ANALYSIS RESULTS(kips-in-unit)

CYCLE NUMBER	CONCRETE STRAIN	N.A. DEPTH (in)	STEEL STRAIN	MOMENT (kipsin)	CURVATURE (1/in)
1	0.000100	19.53	-0.000001	432.13	0.0000051
2	0.000200	12.34	-0.000121	833.64	0.0000162
3	0.000300	10.27	-0.000279	1160.99	0.0000292
4	0.000400	9.31	-0.000451	1484.96	0.0000429
5	0.000500	8.78	-0.000629	1810.21	0.0000570
6	0.000600	8.43	-0.000809	2136.06	0.0000711
7	0.000700	8.20	-0.000991	2461.86	0.0000854
8	0.000800	8.03	-0.001174	2787.14	0.0000997
9	0.000900	7.90	-0.001357	3110.69	0.0001139
10	0.001000	7.80	-0.001539	3431.36	0.0001282
11	0.001100	7.73	-0.001720	3748.94	0.0001424
12	0.001200	7.67	-0.001900	4062.21	0.0001565
13	0.001300	7.62	-0.002078	4370.60	0.0001705
14	0.001400	7.59	-0.002254	4673.42	0.0001845
15	0.001500	7.52	-0.002450	4909.76	0.0001994
16	0.001600	7.44	-0.002658	5106.71	0.0002150
17	0.001800	7.27	-0.003103	5400.46	0.0002475
18	0.002000	7.11	-0.003572	5609.23	0.0002813
19	0.002500	6.80	-0.004787	5915.95	0.0003679
20	0.003000	6.62	-0.005981	6043.06	0.0004534
21	0.003500	6.53	-0.007120	6076.17	0.0005361
22	0.004000	6.51	-0.008172	6083.35	0.0006145
23	0.004500	6.53	-0.009145	6082.76	0.0006888
24	0.005000	6.58	-0.010054	6067.13	0.0007600
25	0.006000	6.71	-0.011715	6005.03	0.0008943
26	0.007000	6.97	-0.012907	5840.29	0.0010050
27	0.008000	7.17	-0.014097	5717.73	0.0011155
28	0.009000	7.35	-0.015252	5620.40	0.0012243
29	0.010000	7.51	-0.016369	5543.09	0.0013312
30	0.012000	7.73	-0.018750	5496.25	0.0015523

CONCRETE STRAIN EXCEEDS MAXIMUM

CALCULATED IDEAL MOMENT 6083 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.01140
80% OF MAXIMUM MOMENT : 4866.7 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL

APPLIED AXIAL LOAD : 100.0 (kips)

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CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER

FILE NAME : N3 DOWELL CONNECTION, P=0.5kips, W11@6"

MOMENT CURVATURE ANALYSIS RESULTS(kips-in-unit)

CYCLE NUMBER	CONCRETE STRAIN	N.A. DEPTH (in)	STEEL STRAIN	MOMENT (kipsin)	CURVATURE (1/in)
1	0.000100	6.91	-0.000187	334.19	0.0000145
2	0.000200	6.89	-0.000375	667.78	0.0000290
3	0.000300	6.89	-0.000563	1001.24	0.0000436
4	0.000400	6.88	-0.000751	1334.42	0.0000581
5	0.000500	6.88	-0.000939	1667.09	0.0000726
6	0.000600	6.88	-0.001126	1998.97	0.0000872
7	0.000700	6.89	-0.001313	2329.65	0.0001016
8	0.000800	6.89	-0.001500	2658.70	0.0001161
9	0.000900	6.90	-0.001686	2985.55	0.0001305
10	0.001000	6.90	-0.001870	3309.63	0.0001449
11	0.001100	6.91	-0.002054	3630.25	0.0001592
12	0.001200	6.92	-0.002235	3946.71	0.0001734
13	0.001300	6.89	-0.002437	4204.07	0.0001887
14	0.001400	6.84	-0.002657	4413.45	0.0002048
15	0.001500	6.77	-0.002891	4583.05	0.0002217
16	0.001600	6.69	-0.003136	4725.81	0.0002391
17	0.001800	6.54	-0.003650	4946.72	0.0002751
18	0.002000	6.40	-0.004186	5108.14	0.0003123
19	0.002500	6.14	-0.005562	5348.30	0.0004070
20	0.003000	6.00	-0.006909	5449.31	0.0005003
21	0.003500	5.94	-0.008166	5500.31	0.0005890
22	0.004000	5.95	-0.009320	5536.68	0.0006724
23	0.004500	5.99	-0.010384	5556.47	0.0007514
24	0.005000	6.05	-0.011384	5558.19	0.0008271
25	0.006000	6.19	-0.013197	5527.43	0.0009691
26	0.007000	6.40	-0.014668	5434.41	0.0010939
27	0.008000	6.62	-0.015940	5339.16	0.0012086
28	0.009000	6.80	-0.017200	5255.15	0.0013226
29	0.010000	6.96	-0.018473	5211.47	0.0014374
30	0.012000	7.18	-0.021111	5178.11	0.0016715

CONCRETE STRAIN EXCEEDS MAXIMUM

CALCULATED IDEAL MOMENT 5558 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.01140
80% OF MAXIMUM MOMENT : 4446.6 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL

APPLIED AXIAL LOAD : 0.5 (kips)

→

CIRCULAR COLUMNS - MANDER
CIRCULAR COLUMNS - MANDER
========================

FILE NAME : N3 DOWELL CONNECTION, P=200kips, W11@6"

MOMENT CURVATURE ANALYSIS RESULTS(kips-in-unit)

CYCLE NUMBER	CONCRETE STRAIN	N.A. DEPTH (in)	STEEL STRAIN	MOMENT (kipsin)	CURVATURE (1/in)
1	0.000100	57.97	0.000066	155.19	0.0000017
2	0.000200	19.53	-0.000003	864.92	0.0000102
3	0.000300	14.59	-0.000107	1312.46	0.0000206
4	0.000400	12.35	-0.000242	1664.66	0.0000324
5	0.000500	11.08	-0.000394	1993.49	0.0000451
6	0.000600	10.28	-0.000556	2315.82	0.0000583
7	0.000700	9.74	-0.000724	2635.97	0.0000719
8	0.000800	9.34	-0.000896	2954.86	0.0000856
9	0.000900	9.05	-0.001071	3274.08	0.0000995
10	0.001000	8.82	-0.001245	3587.20	0.0001134
11	0.001100	8.64	-0.001421	3899.90	0.0001273
12	0.001200	8.50	-0.001596	4208.31	0.0001411
13	0.001300	8.39	-0.001770	4511.69	0.0001550
14	0.001400	8.30	-0.001942	4809.34	0.0001687
15	0.001500	8.23	-0.002112	5100.61	0.0001824
16	0.001600	8.17	-0.002281	5383.07	0.0001959
17	0.001800	7.99	-0.002664	5774.29	0.0002254
18	0.002000	7.81	-0.003075	6045.31	0.0002562
19	0.002500	7.45	-0.004150	6428.33	0.0003357
20	0.003000	7.23	-0.005217	6586.05	0.0004148
21	0.003500	7.12	-0.006231	6630.91	0.0004913
22	0.004000	7.08	-0.007190	6615.37	0.0005649
23	0.004500	7.08	-0.008085	6585.85	0.0006353
24	0.005000	7.12	-0.008916	6554.02	0.0007025
25	0.006000	7.28	-0.010332	6424.39	0.0008245
26	0.007000	7.51	-0.011452	6225.83	0.0009315
27	0.008000	7.73	-0.012493	6057.44	0.0010345
28	0.009000	7.90	-0.013574	5949.69	0.0011396
29	0.010000	8.08	-0.014525	5831.40	0.0012381
30	0.012000	8.29	-0.016672	5765.79	0.0014474

CONCRETE STRAIN EXCEEDS MAXIMUM

CALCULATED IDEAL MOMENT 6631 (kipsin) EQUALS MAXIMUM MOMENT

MAXIMUM NUMBER OF CYCLES : 36
MAXIMUM CONCRETE STRAIN : 0.01140
80% OF MAXIMUM MOMENT : 5304.7 (kipsin)
STEEL LIMIT STRAIN : 0.120000
ITERATION ACCURACY : 0.001
STRAIN HARDENING OF STEEL

APPLIED AXIAL LOAD : 200.0 (kips)

→

```
PUSHOVER ANALYSIS F-LINE SIMULATION
2 0 1 0 0 -1
36 35 19 1 1 2 386.2 5. 5. 0.01 5.8 1.
20 10 0 1 1.0 10 10 1
2
NODES 3
1 0 -500 1 1 1
2 0 -400
3 10 -400 1 1 1
4 0 -300
5 10 -300 1 1 1
6 0 -250
7 10 -250 1 1 1
8 0 -200
9 10 -200 1 1 1
10 0 -150
11 10 -150 1 1 1
12 0 -126
13 10 -126 1 1 1
14 0 -102 0 0 0 0 0 0 0
15 10 -102 1 1 1
16 0 -90
17 10 -90 1 1 1
18 0 -78
19 10 -78 1 1 1
20 0 -66 0 0 0 0 0 0 0
21 10 -66 1 1 1
22 0 -54
23 10 -54 1 1 1
24 0 -42 0 0 0 0 0 0 0
25 10 -42 1 1 1
26 0 -30
27 10 -30 1 1 1
28 0 -18
29 10 -18 1 1 1
30 0 -6 0 0 0 0 0 0 0
31 10 -6 1 1 1
32 0 8
33 0 16
34 0 24
35 0 36.6 0 1 1 0 0 0 0
36 10 36.6 1 1 1
ELEMENTS 3
1 1 1 2
2 1 2 4
3 1 4 6
4 1 6 8
5 1 8 10
6 1 10 12 0 0 0
7 1 12 14 0 0 0
8 1 14 16 0 0 0
9 1 16 18 0 0 0
10 1 18 20 0 0 0
11 1 20 22 0 0 0
12 1 22 24 0 0 0
13 1 24 26 0 0 0
14 1 26 28 0 0 0
15 1 28 30 0 0 0
16 2 30 32 0 0 0
17 2 32 33 0 0 0
```

F-LINE PUSHOVER
INPUT DATA
3pages

```
18 2 33 34 0 0 0
19 3 34 35
20 4 2 3
21 5 4 5
22 6 6 7
23 7 8 9
24 8 10 11
25 9 12 13
26 10 14 15
27 11 16 17
28 12 18 19
29 13 20 21
30 14 22 23
31 15 24 25
32 16 26 27
33 17 28 29
34 18 30 31
35 19 35 36
PROPS
1 FRAME
1 0 0 13
4770 2000 452 362 16286 0
1 0 10.0 10.0
500 -500 6217 -6217 6217 -6217
0.252 2650 -2650 2650 -2650
2 FRAME
1 0 0 2
4770 2000 452 115 5100 0.0
1 0.0 10.0 10.0
500 -500 5692 -5692 5692 -5692
3 FRAME
1 0 0 0
4770 2000 452 0.0 5100 0.0
4 SPRING
1 0 0 0 3333 0. 0. 0.001
5 SPRING
1 0 0 0 1875 0. 0. 0.001
6 SPRING
1 0 0 0 1042 0. 0. 0.001
7 SPRING
1 0 0 0 833 0. 0. 0.001
8 SPRING
1 0 0 0 463 0. 0. 0.001
9 SPRING
1 2 0 0 252 0. 0. 0.001 0.001
126 -126 126 -126 126 -126
10 SPRING
1 2 0 0 153 0. 0. 0.001 0.001
76.5 -76.5 76.5 -76.5 76.5 -76.5
11 SPRING
1 2 0 0 90 0. 0. 0.001 0.001
45 -45 45 -45 45 -45
12 SPRING
1 2 0 0 78 0. 0. 0.001 0.001
39 -39 39 -39 39 -39
13 SPRING
1 2 0 0 66 0. 0. 0.001 0.001
33 -33 33 -33 33 -33
14 SPRING
1 2 0 0 54 0. 0. 0.001 0.001
```

```
27 -27 27 -27 27 -27
15 SPRING
1 2 0 0 42 0. 0. 0.001 0.001
21 -21 21 -21 21 -21
16 SPRING
1 2 0 0 30 0. 0. 0.001  0.001
15 -15 15 -15 15 -15
17 SPRING
1 2 0 0 18 0. 0. 0.001  0.001
9 -9 9 -9 9 -9
18 SPRING
1 2 0 0 6 0. 0. 0.001 0.001
3 -3 3 -3 3 -3
19 SPRING
1 0 0 0 50 0. 0. 0.001
WEIGHTS 0
35 1. 1. 1.
LOADS
35 0. 0. 0.
SHAPE
35 900. 0. 0.
EQUAKE
3 1 0.1 1.0 0
START
1 0.0 0.0
2 10 1.0
```

DATE: 21 JANUARY 2000 TIME 1:34: 1.83

STRUCTURE

PUSHOVER ANALYSIS F-LINE SIMULATION

ANALYSIS DETAILS

Analysis Type : In-elastic
Time Variation: Time-history - Newmark (Beta = 0.25)
Mass Matrix : Lumped
Damping Matrix: Alpha*[Mass]+Beta*[Initial Stiffness] Rayleigh Damping
Dynamic Force Loading with 1 Time-varying Force Inputs
Binary post-processor (.RES) file
Small Displacement analysis

STRUCTURAL DATA

Number of Space Dimensions	2
Number of Equations per Node	3
Number of Nodes	36
No.Apparent Degrees of Freedom	108
Number of Members	35
Number of Member Sections	19
Number of Mode Shapes Required	1
Pictures of Displaced Frame	NO
Acceleration of Gravity	386.200
% Critical Damping Mode 1 =	5.00
% Critical Damping Mode 2 =	5.00

F-LINE PUSHOVER
DATA ECHO

7pages

TIME-HISTORY DATA

Excitation Time-step	.01000 Seconds
Duration of Excitation	5.800 Seconds
Excitation Multiplier	1.000E+00
No. Steps between Print-outs	20
No. Steps between Disk Output	10
No. Steps between Pictures	0
ScreenPlot Mutiplier	10.000
Max. X Displace.(Screen)	10.000
Max. Y Displace.(Screen)	1.000
Max. Cycles of Newton-Raphson	2
Max. Cycles Damping Iteration	0
Force Norm Limit	1.000E-03
Wave Velocity - X axis.	0.000E+00
Wave Velocity - Y axis.	0.000E+00

DYNAPLOT Nodal Output: Displacement,Damping Force,Inertia Force & Load

1NODE	POSITION OF NODES	NODE FIXITY			MASTER NODE			OUTPUT
No.	X Co-ord Y Co-ord	X	Y	Z	X	Y	Z	Flag
1	0.0000E+00-5.0000E+02	1	1	1	0	0	0	3
2	0.0000E+00-4.0000E+02	0	0	0	0	0	0	3
3	1.0000E+01-4.0000E+02	1	1	1	0	0	0	3
4	0.0000E+00-3.0000E+02	0	0	0	0	0	0	3
5	1.0000E+01-3.0000E+02	1	1	1	0	0	0	3
6	0.0000E+00-2.5000E+02	0	0	0	0	0	0	3
7	1.0000E+01-2.5000E+02	1	1	1	0	0	0	3
8	0.0000E+00-2.0000E+02	0	0	0	0	0	0	3
9	1.0000E+01-2.0000E+02	1	1	1	0	0	0	3

10	0.0000E+00-1.5000E+02	0	0	0	0	0	0	3
11	1.0000E+01-1.5000E+02	1	1	1	0	0	0	3
12	0.0000E+00-1.2600E+02	0	0	0	0	0	0	3
13	1.0000E+01-1.2600E+02	1	1	1	0	0	0	3
14	0.0000E+00-1.0200E+02	0	0	0	0	0	0	0
15	1.0000E+01-1.0200E+02	1	1	1	0	0	0	3
16	0.0000E+00-9.0000E+01	0	0	0	0	0	0	3
17	1.0000E+01-9.0000E+01	1	1	1	0	0	0	3
18	0.0000E+00-7.8000E+01	0	0	0	0	0	0	3
19	1.0000E+01-7.8000E+01	1	1	1	0	0	0	3
20	0.0000E+00-6.6000E+01	0	0	0	0	0	0	0
21	1.0000E+01-6.6000E+01	1	1	1	0	0	0	3
22	0.0000E+00-5.4000E+01	0	0	0	0	0	0	3
23	1.0000E+01-5.4000E+01	1	1	1	0	0	0	3
24	0.0000E+00-4.2000E+01	0	0	0	0	0	0	0
25	1.0000E+01-4.2000E+01	1	1	1	0	0	0	3
26	0.0000E+00-3.0000E+01	0	0	0	0	0	0	3
27	1.0000E+01-3.0000E+01	1	1	1	0	0	0	3
28	0.0000E+00-1.8000E+01	0	0	0	0	0	0	3
29	1.0000E+01-1.8000E+01	1	1	1	0	0	0	3
30	0.0000E+00-6.0000E+00	0	0	0	0	0	0	0
31	1.0000E+01-6.0000E+00	1	1	1	0	0	0	3
32	0.0000E+00 8.0000E+00	0	0	0	0	0	0	3
33	0.0000E+00 1.6000E+01	0	0	0	0	0	0	3
34	0.0000E+00 2.4000E+01	0	0	0	0	0	0	3
35	0.0000E+00 3.6600E+01	0	1	1	0	0	0	0
36	1.0000E+01 3.6600E+01	1	1	1	0	0	0	3

1

MEMBER Number	SECTION Number	NODE End1	NODE End2	NODE Inner1	NODE Inner2	NODE zz-axis	OUTPUT Flag
1	1	1	2	1	2		3
2	1	2	4	2	4		3
3	1	4	6	4	6		3
4	1	6	8	6	8		3
5	1	8	10	8	10		3
6	1	10	12	10	12		0
7	1	12	14	12	14		0
8	1	14	16	14	16		0
9	1	16	18	16	18		0
10	1	18	20	18	20		0
11	1	20	22	20	22		0
12	1	22	24	22	24		0
13	1	24	26	24	26		0
14	1	26	28	26	28		0
15	1	28	30	28	30		0
16	2	30	32	30	32		0
17	2	32	33	32	33		0
18	2	33	34	33	34		0
19	3	34	35	34	35		3
20	4	2	3	2	3		3
21	5	4	5	4	5		3
22	6	6	7	6	7		3
23	7	8	9	8	9		3
24	8	10	11	10	11		3
25	9	12	13	12	13		3
26	10	14	15	14	15		3
27	11	16	17	16	17		3
28	12	18	19	18	19		3
29	13	20	21	20	21		3
30	14	22	23	22	23		3

31	15	24	25	24	25	3
32	16	26	27	26	27	3
33	17	28	29	28	29	3
34	18	30	31	30	31	3
35	19	35	36	35	36	3

1

MEMBER PROPERTIES TABLE
======================

```
0SECTION NUMBER    1 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
   Elastic Modulus . . . . = 4.770E+03     Shear Modulus . . . . . . = 2.000E+03
   Cross Sectional Area. . = 4.520E+02     Shear Area. . . . . . . . = 3.620E+02
   Moment of Inertia . . . = 1.629E+04     Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 0.000E+00     End-block Length  End 2 = 0.000E+00
   Joint Flexibility End 1 = 0.000E+00     Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =        NO     Perfect Hinge  at End 2 =        NO
   Bi-linear r Axial . . . =     1.000     Bi-linear r Flexure . . =      .000
   Plas.Hinge Length End 1 =    10.000     Plas.Hinge Length End 2 =    10.000

   Giberson ONE-COMPONENT BEAM SECTION
   Tension Yield . . . . . = 5.000E+02     Compression Yield . . . =-5.000E+02
   Yield Moment +ve  End 1 = 6.217E+03     Yield Moment -ve  End 1 =-6.217E+03
   Yield Moment +ve  End 2 = 6.217E+03     Yield Moment -ve  End 2 =-6.217E+03

   MUTO Hysteresis (A=End1 or X, B=End2 or Y)
   Cracked Stiffness Fact. =     .2520
   Positive Cracking "A" . = 2.650E+03     Negative Cracking "A" . =-2.650E+03
   Positive Cracking "B" . = 2.650E+03     Negative Cracking "B" . =-2.650E+03

0SECTION NUMBER    2 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
   Elastic Modulus . . . . = 4.770E+03     Shear Modulus . . . . . . = 2.000E+03
   Cross Sectional Area. . = 4.520E+02     Shear Area. . . . . . . . = 1.150E+02
   Moment of Inertia . . . = 5.100E+03     Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 0.000E+00     End-block Length  End 2 = 0.000E+00
   Joint Flexibility End 1 = 0.000E+00     Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =        NO     Perfect Hinge  at End 2 =        NO
   Bi-linear r Axial . . . =     1.000     Bi-linear r Flexure . . =      .000
   Plas.Hinge Length End 1 =    10.000     Plas.Hinge Length End 2 =    10.000

   Giberson ONE-COMPONENT BEAM SECTION
   Tension Yield . . . . . = 5.000E+02     Compression Yield . . . =-5.000E+02
   Yield Moment +ve  End 1 = 5.692E+03     Yield Moment -ve  End 1 =-5.692E+03
   Yield Moment +ve  End 2 = 5.692E+03     Yield Moment -ve  End 2 =-5.692E+03

   Bi-Linear Hysteresis

0SECTION NUMBER    3 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
   Elastic Modulus . . . . = 4.770E+03     Shear Modulus . . . . . . = 2.000E+03
   Cross Sectional Area. . = 4.520E+02     Shear Area. . . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.100E+03     Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 0.000E+00     End-block Length  End 2 = 0.000E+00
   Joint Flexibility End 1 = 0.000E+00     Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =        NO     Perfect Hinge  at End 2 =        NO

   Linear Elastic Hysteresis

0SECTION NUMBER    4 TYPE=SPRING
 ------------------------------
   Spring Stiffness in X . = 3.333E+03     Spring Stiffness in Y . = 0.000E+00
```

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Rotational Stiffness. . = 0.000E+00      Weight/(Unit Length). . = 1.000E-03
Bilinear Factor Transln.=      .000      Bilinear Factor Rotation=      .000
Angle Global-Local axes =      .000

Linear Elastic Hysteresis

0SECTION NUMBER    5 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 1.875E+03    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .000    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  Linear Elastic Hysteresis

0SECTION NUMBER    6 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 1.042E+03    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .000    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  Linear Elastic Hysteresis

0SECTION NUMBER    7 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 8.330E+02    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .000    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  Linear Elastic Hysteresis

0SECTION NUMBER    8 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 4.630E+02    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .000    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  Linear Elastic Hysteresis

0SECTION NUMBER    9 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 2.520E+02    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .001    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
  Positive X Force. . . . = 1.260E+02    Negative X Force. . . . =-1.260E+02
  Positive Y Force. . . . = 1.260E+02    Negative Y Force. . . . =-1.260E+02
  Positive Moment . . . . = 1.260E+02    Negative Moment . . . . =-1.260E+02

  Bi-Linear Hysteresis

0SECTION NUMBER   10 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 1.530E+02    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
```

```
    Bilinear Factor Transln.=       .001    Bilinear Factor Rotation=       .000
    Angle Global-Local axes =       .000

    3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
    Positive X Force. . . . = 7.650E+01    Negative X Force. . . . =-7.650E+01
    Positive Y Force. . . . = 7.650E+01    Negative Y Force. . . . =-7.650E+01
    Positive Moment . . . . = 7.650E+01    Negative Moment . . . . =-7.650E+01

    Bi-Linear Hysteresis

0SECTION NUMBER  11 TYPE=SPRING
 ------------------------------
    Spring Stiffness in X . = 9.000E+01    Spring Stiffness in Y . = 0.000E+00
    Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
    Bilinear Factor Transln.=       .001    Bilinear Factor Rotation=       .000
    Angle Global-Local axes =       .000

    3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
    Positive X Force. . . . = 4.500E+01    Negative X Force. . . . =-4.500E+01
    Positive Y Force. . . . = 4.500E+01    Negative Y Force. . . . =-4.500E+01
    Positive Moment . . . . = 4.500E+01    Negative Moment . . . . =-4.500E+01

    Bi-Linear Hysteresis

0SECTION NUMBER  12 TYPE=SPRING
 ------------------------------
    Spring Stiffness in X . = 7.800E+01    Spring Stiffness in Y . = 0.000E+00
    Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
    Bilinear Factor Transln.=       .001    Bilinear Factor Rotation=       .000
    Angle Global-Local axes =       .000

    3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
    Positive X Force. . . . = 3.900E+01    Negative X Force. . . . =-3.900E+01
    Positive Y Force. . . . = 3.900E+01    Negative Y Force. . . . =-3.900E+01
    Positive Moment . . . . = 3.900E+01    Negative Moment . . . . =-3.900E+01

    Bi-Linear Hysteresis

0SECTION NUMBER  13 TYPE=SPRING
 ------------------------------
    Spring Stiffness in X . = 6.600E+01    Spring Stiffness in Y . = 0.000E+00
    Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
    Bilinear Factor Transln.=       .001    Bilinear Factor Rotation=       .000
    Angle Global-Local axes =       .000

    3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
    Positive X Force. . . . = 3.300E+01    Negative X Force. . . . =-3.300E+01
    Positive Y Force. . . . = 3.300E+01    Negative Y Force. . . . =-3.300E+01
    Positive Moment . . . . = 3.300E+01    Negative Moment . . . . =-3.300E+01

    Bi-Linear Hysteresis

0SECTION NUMBER  14 TYPE=SPRING
 ------------------------------
    Spring Stiffness in X . = 5.400E+01    Spring Stiffness in Y . = 0.000E+00
    Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
    Bilinear Factor Transln.=       .001    Bilinear Factor Rotation=       .000
    Angle Global-Local axes =       .000

    3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
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Positive X Force. . . . = 2.700E+01      Negative X Force. . . . =-2.700E+01
Positive Y Force. . . . = 2.700E+01      Negative Y Force. . . . =-2.700E+01
Positive Moment . . . . = 2.700E+01      Negative Moment . . . . =-2.700E+01

Bi-Linear Hysteresis

0SECTION NUMBER   15 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 4.200E+01    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .001    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
  Positive X Force. . . . = 2.100E+01    Negative X Force. . . . =-2.100E+01
  Positive Y Force. . . . = 2.100E+01    Negative Y Force. . . . =-2.100E+01
  Positive Moment . . . . = 2.100E+01    Negative Moment . . . . =-2.100E+01

  Bi-Linear Hysteresis

0SECTION NUMBER   16 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 3.000E+01    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .001    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
  Positive X Force. . . . = 1.500E+01    Negative X Force. . . . =-1.500E+01
  Positive Y Force. . . . = 1.500E+01    Negative Y Force. . . . =-1.500E+01
  Positive Moment . . . . = 1.500E+01    Negative Moment . . . . =-1.500E+01

  Bi-Linear Hysteresis

0SECTION NUMBER   17 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 1.800E+01    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .001    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
  Positive X Force. . . . = 9.000E+00    Negative X Force. . . . =-9.000E+00
  Positive Y Force. . . . = 9.000E+00    Negative Y Force. . . . =-9.000E+00
  Positive Moment . . . . = 9.000E+00    Negative Moment . . . . =-9.000E+00

  Bi-Linear Hysteresis

0SECTION NUMBER   18 TYPE=SPRING
 ------------------------------
  Spring Stiffness in X . = 6.000E+00    Spring Stiffness in Y . = 0.000E+00
  Rotational Stiffness. . = 0.000E+00    Weight/(Unit Length). . = 1.000E-03
  Bilinear Factor Transln.=      .001    Bilinear Factor Rotation=      .000
  Angle Global-Local axes =      .000

  3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
  Positive X Force. . . . = 3.000E+00    Negative X Force. . . . =-3.000E+00
  Positive Y Force. . . . = 3.000E+00    Negative Y Force. . . . =-3.000E+00
  Positive Moment . . . . = 3.000E+00    Negative Moment . . . . =-3.000E+00
```

Bi-Linear Hysteresis

0SECTION NUMBER 19 TYPE=SPRING

Spring Stiffness in X . =	5.000E+01	Spring Stiffness in Y . =	0.000E+00
Rotational Stiffness. . =	0.000E+00	Weight/(Unit Length). . =	1.000E-03
Bilinear Factor Transln.=	.000	Bilinear Factor Rotation=	.000
Angle Global-Local axes =	.000		

Linear Elastic Hysteresis

NAI.DAT A line Data

PUSHOVER RESULTS,
A-LINE PILE
(DISP, in.) (Force, kips).

Horizontal Axis=DI
Vertical Δ Axis=F

	Δ	F
1	0.0000E+00	0.0000E+00
2	1.7714E-01	2.4290E-01
3	3.5244E-01	4.8327E-01
4	5.2700E-01	7.2263E-01
5	7.0140E-01	9.6178E-01
6	8.7590E-01	1.2011E+00
7	1.0506E+00	1.4406E+00
8	1.2254E+00	1.6804E+00
9	1.4004E+00	1.9203E+00
10	1.5755E+00	2.1604E+00
11	1.7507E+00	2.4006E+00
12	1.9259E+00	2.6408E+00
13	2.1011E+00	2.8810E+00
14	2.2763E+00	3.1213E+00
15	2.4515E+00	3.3615E+00
16	2.6267E+00	3.6017E+00
17	2.8019E+00	3.8420E+00
18	2.9771E+00	4.0822E+00
19	3.1523E+00	4.3225E+00
20	3.3275E+00	4.5627E+00
21	3.5027E+00	4.8029E+00
22	3.6779E+00	5.0431E+00
23	3.8531E+00	5.2834E+00
24	4.0282E+00	5.5236E+00
25	4.2034E+00	5.7638E+00
26	4.3786E+00	6.0041E+00
27	4.5538E+00	6.2443E+00
28	4.7290E+00	6.4845E+00
29	4.9042E+00	6.7248E+00
30	5.0794E+00	6.9650E+00
31	5.2546E+00	7.2052E+00
32	5.4298E+00	7.4455E+00
33	5.6050E+00	7.6857E+00
34	5.7802E+00	7.9259E+00
35	5.9554E+00	8.1662E+00
36	6.1306E+00	8.4064E+00
37	6.3060E+00	8.6345E+00
38	6.4822E+00	8.8267E+00
39	6.6586E+00	9.0049E+00
40	6.8353E+00	9.1720E+00
41	7.0119E+00	9.3398E+00
42	7.1886E+00	9.5077E+00
43	7.3652E+00	9.6730E+00
44	7.5420E+00	9.8362E+00
45	7.7187E+00	9.9994E+00
46	7.8956E+00	1.0153E+01
47	8.0727E+00	1.0301E+01
48	8.2497E+00	1.0449E+01
49	8.4268E+00	1.0598E+01
50	8.6038E+00	1.0745E+01
51	8.7809E+00	1.0891E+01
52	8.9580E+00	1.1037E+01
53	9.1351E+00	1.1184E+01
54	9.3121E+00	1.1330E+01
55	9.4892E+00	1.1477E+01
56	9.6663E+00	1.1624E+01
57	9.8433E+00	1.1771E+01
58	1.0020E+01	1.1918E+01
	(Δ)	(F)

NB1.DAT B Line Data

PUSHOVER RESULTS

B-LINE PILE

(DISP, ins) (Force, kips)

Horizontal Axis=DISP
Vertical Δ Axis=FORCE F

1	0.0000E+00	0.0000E+00
2	1.7328E-01	4.0315E-01
3	3.4533E-01	8.0345E-01
4	5.1690E-01	1.2026E+00
5	6.8836E-01	1.6015E+00
6	8.5987E-01	2.0006E+00
7	1.0315E+00	2.3999E+00
8	1.2032E+00	2.7994E+00
9	1.3750E+00	3.1991E+00
10	1.5469E+00	3.5990E+00
11	1.7188E+00	3.9990E+00
12	1.8908E+00	4.3991E+00
13	2.0628E+00	4.7992E+00
14	2.2347E+00	5.1994E+00
15	2.4067E+00	5.5996E+00
16	2.5787E+00	5.9997E+00
17	2.7507E+00	6.3999E+00
18	2.9227E+00	6.8001E+00
19	3.0947E+00	7.2002E+00
20	3.2667E+00	7.6004E+00
21	3.4387E+00	8.0006E+00
22	3.6107E+00	8.4007E+00
23	3.7827E+00	8.8009E+00
24	3.9547E+00	9.2011E+00
25	4.1267E+00	9.6013E+00
26	4.2987E+00	1.0001E+01
27	4.4707E+00	1.0402E+01
28	4.6434E+00	1.0768E+01
29	4.8172E+00	1.1078E+01
30	4.9913E+00	1.1371E+01
31	5.1657E+00	1.1649E+01
32	5.3403E+00	1.1919E+01
33	5.5149E+00	1.2190E+01
34	5.6895E+00	1.2460E+01
35	5.8641E+00	1.2731E+01
36	6.0389E+00	1.2988E+01
37	6.2141E+00	1.3230E+01
38	6.3892E+00	1.3472E+01
39	6.5644E+00	1.3715E+01
40	6.7395E+00	1.3958E+01
41	6.9146E+00	1.4202E+01
42	7.0898E+00	1.4446E+01
43	7.2649E+00	1.4690E+01
44	7.4400E+00	1.4934E+01
45	7.6152E+00	1.5176E+01
46	7.7903E+00	1.5417E+01
47	7.9655E+00	1.5659E+01
48	8.1407E+00	1.5900E+01
49	8.3158E+00	1.6142E+01
50	8.4911E+00	1.6380E+01
51	8.6663E+00	1.6618E+01
52	8.8415E+00	1.6856E+01
53	9.0168E+00	1.7095E+01
54	9.1920E+00	1.7333E+01
55	9.3672E+00	1.7571E+01
56	9.5425E+00	1.7810E+01
57	9.7177E+00	1.8049E+01
58	9.8929E+00	1.8287E+01
	(Δ)	(F)

NC1.DH1 . C Line Data.

Horizontal Axis=D
Vertical Δ Axis=F

		F
1	0.0000E+00	0.0000E+00
2	1.5393E-01	6.8562E-01
3	3.1913E-01	1.4214E+00
4	4.8441E-01	2.1575E+00
5	6.4969E-01	2.8936E+00
6	8.1496E-01	3.6297E+00
7	9.8024E-01	4.3658E+00
8	1.1455E+00	5.1019E+00
9	1.3108E+00	5.8380E+00
10	1.4761E+00	6.5741E+00
11	1.6414E+00	7.3102E+00
12	1.8066E+00	8.0463E+00
13	1.9719E+00	8.7824E+00
14	2.1372E+00	9.5186E+00
15	2.3025E+00	1.0255E+01
16	2.4677E+00	1.0991E+01
17	2.6330E+00	1.1727E+01
18	2.7983E+00	1.2463E+01
19	2.9636E+00	1.3199E+01
20	3.1289E+00	1.3930E+01
21	3.2969E+00	1.4523E+01
22	3.4660E+00	1.5064E+01
23	3.6355E+00	1.5587E+01
24	3.8055E+00	1.6087E+01
25	3.9756E+00	1.6582E+01
26	4.1458E+00	1.7070E+01
27	4.3161E+00	1.7558E+01
28	4.4867E+00	1.8023E+01
29	4.6576E+00	1.8476E+01
30	4.8285E+00	1.8929E+01
31	4.9996E+00	1.9376E+01
32	5.1707E+00	1.9823E+01
33	5.3417E+00	2.0271E+01
34	5.5128E+00	2.0716E+01
35	5.6840E+00	2.1156E+01
36	5.8552E+00	2.1596E+01
37	6.0264E+00	2.2037E+01
38	6.1976E+00	2.2477E+01
39	6.3687E+00	2.2918E+01
40	6.5400E+00	2.3355E+01
41	6.7113E+00	2.3792E+01
42	6.8825E+00	2.4229E+01
43	7.0539E+00	2.4660E+01
44	7.2255E+00	2.5081E+01
45	7.3970E+00	2.5502E+01
46	7.5686E+00	2.5923E+01
47	7.7402E+00	2.6344E+01
48	7.9119E+00	2.6719E+01
49	8.0893E+00	2.6869E+01
50	8.2668E+00	2.6995E+01
51	8.4443E+00	2.7120E+01
52	8.6218E+00	2.7246E+01
53	8.7993E+00	2.7372E+01
54	8.9767E+00	2.7498E+01
55	9.1542E+00	2.7623E+01
56	9.3317E+00	2.7749E+01
57	9.5092E+00	2.7875E+01
58	9.6867E+00	2.8000E+01
	(Δ)	(F)

PUSHOVER RESULTS
C-LINE PILE
(DISP, ins) (Force, kips)

(NDI) D line data. Level ground.

PUSHOVER RESULTS

D-LINE PILE

(DISP, ins) (Force, kips)

(Note: -ve sign has no significance)

Horizontal Axis=DISP
Vertical Δ Axis=FORCE F

1	0.0000E+00	0.0000E+00
2	1.4605E-01	-1.5146E+00
3	2.9452E-01	-3.0543E+00
4	4.4408E-01	-4.6052E+00
5	5.9393E-01	-6.1591E+00
6	7.4368E-01	-7.7120E+00
7	8.9321E-01	-9.2627E+00
8	1.0425E+00	-1.0811E+01
9	1.1917E+00	-1.2358E+01
10	1.3408E+00	-1.3905E+01
11	1.4899E+00	-1.5450E+01
12	1.6390E+00	-1.6996E+01
13	1.7880E+00	-1.8542E+01
14	1.9371E+00	-2.0088E+01
15	2.0920E+00	-2.1354E+01
16	2.2484E+00	-2.2524E+01
17	2.4063E+00	-2.3624E+01
18	2.5643E+00	-2.4721E+01
19	2.7227E+00	-2.5804E+01
20	2.8817E+00	-2.6851E+01
21	3.0408E+00	-2.7898E+01
22	3.2007E+00	-2.8899E+01
23	3.3613E+00	-2.9868E+01
24	3.5219E+00	-3.0839E+01
25	3.6824E+00	-3.1815E+01
26	3.8429E+00	-3.2790E+01
27	4.0037E+00	-3.3751E+01
28	4.1653E+00	-3.4672E+01
29	4.3269E+00	-3.5594E+01
30	4.4884E+00	-3.6515E+01
31	4.6500E+00	-3.7437E+01
32	4.8174E+00	-3.8098E+01
33	4.9915E+00	-3.8379E+01
34	5.1656E+00	-3.8661E+01
35	5.3398E+00	-3.8945E+01
36	5.5143E+00	-3.9215E+01
37	5.6891E+00	-3.9476E+01
38	5.8639E+00	-3.9738E+01
39	6.0386E+00	-3.9999E+01
40	6.2134E+00	-4.0261E+01
41	6.3882E+00	-4.0522E+01
42	6.5629E+00	-4.0784E+01
43	6.7377E+00	-4.1045E+01
44	6.9125E+00	-4.1307E+01
45	7.0873E+00	-4.1568E+01
46	7.2620E+00	-4.1830E+01
47	7.4368E+00	-4.2091E+01
48	7.6116E+00	-4.2353E+01
49	7.7863E+00	-4.2614E+01
50	7.9611E+00	-4.2875E+01
51	8.1360E+00	-4.3133E+01
52	8.3113E+00	-4.3370E+01
53	8.4866E+00	-4.3603E+01
54	8.6615E+00	-4.3825E+01
55	8.8369E+00	-4.4071E+01
56	9.0161E+00	-4.4113E+01
57	9.1962E+00	-4.4124E+01
58	9.3763E+00	-4.4116E+01
	(Δ)	(F)

E line data. Level ground.

Horizontal Axis=DISP
Vertical Δ Axis=FORCE F

1	0.0000E+00	0.0000E+00
2	1.0242E-01	-3.5537E+00
3	2.1373E-01	-7.4148E+00
4	3.1831E-01	-1.1043E+01
5	4.2375E-01	-1.4701E+01
6	5.3115E-01	-1.8427E+01
7	6.3703E-01	-2.2100E+01
8	7.4312E-01	-2.5781E+01
9	8.4964E-01	-2.9476E+01
10	9.5581E-01	-3.3160E+01
11	1.0622E+00	-3.6825E+01
12	1.1789E+00	-3.9993E+01
13	1.2992E+00	-4.2983E+01
14	1.4199E+00	-4.5942E+01
15	1.5434E+00	-4.8769E+01
16	1.6670E+00	-5.1594E+01
17	1.7923E+00	-5.4326E+01
18	1.9185E+00	-5.7020E+01
19	2.0446E+00	-5.9711E+01
20	2.1717E+00	-6.2357E+01
21	2.3015E+00	-6.4868E+01
22	2.4521E+00	-6.6276E+01
23	2.6148E+00	-6.7165E+01
24	2.7775E+00	-6.8042E+01
25	2.9401E+00	-6.8917E+01
26	3.1043E+00	-6.9757E+01
27	3.2680E+00	-7.0432E+01
28	3.4341E+00	-7.1272E+01
29	3.5987E+00	-7.1951E+01
30	3.7628E+00	-7.2733E+01
31	3.9275E+00	-7.3481E+01
32	4.0934E+00	-7.4208E+01
33	4.2592E+00	-7.4950E+01
34	4.4256E+00	-7.5698E+01
35	4.5904E+00	-7.6427E+01
36	4.7569E+00	-7.7093E+01
37	4.9239E+00	-7.7672E+01
38	5.1059E+00	-7.7636E+01
39	5.2850E+00	-7.7684E+01
40	5.4628E+00	-7.7706E+01
41	5.6494E+00	-7.7544E+01
42	5.8237E+00	-7.7589E+01
43	6.0057E+00	-7.7481E+01
44	6.1833E+00	-7.7678E+01
45	6.3663E+00	-7.7404E+01
46	6.5433E+00	-7.7480E+01
47	6.7283E+00	-7.7488E+01
48	6.9045E+00	-7.7679E+01
49	7.0866E+00	-7.7386E+01
50	7.2616E+00	-7.7685E+01
51	7.4448E+00	-7.7694E+01
52	7.6242E+00	-7.7683E+01
53	7.8079E+00	-7.7693E+01
54	7.9858E+00	-7.7697E+01
55	8.1662E+00	-7.7696E+01
56	8.3449E+00	-7.7703E+01
57	8.5265E+00	-7.7693E+01
58	8.7045E+00	-7.7698E+01
	(Δ)	(F)

PUSHOVER RESULTS
E-LINE PILE
(DISP, ins) (Force, kips)

(Note: -ve sign has no significance)

F Line Data. Level ground.

PUSHOVER RESULTS

F-LINE PILE

(Disp, ins) (Force, kips)

~~Note:~~

Horizontal Axis=D
Vertical Axis=F

	Δ	F
1	0.0000E+00	0.0000E+00
2	4.9029E-02	6.6069E+00
3	9.7748E-02	1.3173E+01
4	1.4635E-01	1.9722E+01
5	1.9493E-01	2.6269E+01
6	2.4353E-01	3.2819E+01
7	2.9216E-01	3.9373E+01
8	3.4082E-01	4.5931E+01
9	3.8951E-01	5.2492E+01
10	4.3820E-01	5.9055E+01
11	4.8691E-01	6.5619E+01
12	5.3562E-01	7.2184E+01
13	5.8904E-01	7.8497E+01
14	6.4521E-01	8.4686E+01
15	7.0284E-01	9.0814E+01
16	7.6383E-01	9.6763E+01
17	8.2637E-01	1.0263E+02
18	8.9178E-01	1.0837E+02
19	9.5912E-01	1.1399E+02
20	1.0268E+00	1.1960E+02
21	1.1476E+00	1.2246E+02
22	1.2729E+00	1.2526E+02
23	1.3982E+00	1.2800E+02
24	1.5310E+00	1.3037E+02
25	1.6647E+00	1.3268E+02
26	1.7992E+00	1.3494E+02
27	1.9347E+00	1.3713E+02
28	2.0716E+00	1.3925E+02
29	2.2134E+00	1.4113E+02
30	2.3550E+00	1.4301E+02
31	2.4966E+00	1.4490E+02
32	2.6383E+00	1.4679E+02
33	2.7967E+00	1.4815E+02
34	2.9691E+00	1.4824E+02
35	3.1451E+00	1.4835E+02
36	3.3222E+00	1.4851E+02
37	3.5018E+00	1.4857E+02
38	3.6817E+00	1.4859E+02
39	3.8616E+00	1.4860E+02
40	4.0415E+00	1.4863E+02
41	4.2213E+00	1.4867E+02
42	4.4012E+00	1.4870E+02
43	4.5811E+00	1.4872E+02
44	4.7610E+00	1.4874E+02
45	4.9408E+00	1.4877E+02
46	5.1207E+00	1.4878E+02
47	5.3006E+00	1.4880E+02
48	5.4805E+00	1.4881E+02
49	5.6604E+00	1.4882E+02
50	5.8402E+00	1.4883E+02
51	6.0201E+00	1.4884E+02
52	6.2000E+00	1.4885E+02
53	6.3799E+00	1.4886E+02
54	6.5597E+00	1.4886E+02
55	6.7396E+00	1.4887E+02
56	6.9195E+00	1.4888E+02
57	7.0994E+00	1.4889E+02
58	7.2792E+00	1.4889E+02
	(Δ)	(F)

```
METHOD B ANALYSIS OF NAVY WHARF
2 0 1 0 0 0
9 12 6 5 1 2 386.2 5. 5. 0.01 2 1.
5 2 0 1 1. 10 5. 5.
2
NODES 0
1 -1386  570.5 0 0 0
2 -1396  570.5 1 1 1
3  1386  570.5 0 0 0
4  1376  570.5 1 1 1
5 -1386 -251.3 0 0 0
6 -1396 -251.3 1 1 1
7  1386 -251.3 0 0 0
8  1376 -251.3 1 1 1
9 0. 0. 0 0 0
ELEMENTS 0
1 1 1 2
2 1 3 4
3 2 5 6
4 2 7 8
5 5 1 3
6 5 5 7
7 3 1 5
8 3 3 7
9 6 1 9
10 6 3 9
11 4 5 9
12 4 7 9
PROPS
1 SPRING
1 4 0 0 2800. 2800. 10000. 0. 0.1 1.
350 -350 350 -350 350 -350
0.5 0.5 1 2
2 SPRING
1 4 0 0 81.5 81.5 1000. 0. 0.1 1.
40 -40 40 -40 40 -40
0.5 0.5 1 2
3 FRAME
1 0 0 0
47700. 20000. 100000. 0. 500000. 0. 400. 400.
4 FRAME
1
47700. 20000. 100000. 0. 500000. 0. 700. 700.
5 FRAME
1
47700.  20000. 100000. 0. 500000. 0. 1350 1350
6 FRAME
1
47700. 20000. 100000. 0. 500000. 0. 745. 745.
WEIGHTS 0
1 3061 3061 3061
3 3061 3061 3061
5 7163 7163 7163
7 7163 7163 7163
LOADS
1 0. 0. 0.
EQUAKE
0 1 0.01 1.0 -1
START
```

$\boxed{\text{METHOD B INPUT DATA}}$

–1 page.

DATE: 15 FEBRUARY 2000 TIME 7:12:37.89

STRUCTURE

METHOD B ANALYSIS OF NAVY WHARF

ANALYSIS DETAILS

```
  Analysis Type : In-elastic
  Time Variation: Time-history - Newmark (Beta = 0.25)
  Mass Matrix   : Lumped
  Damping Matrix: Alpha*[Mass]+Beta*[Initial Stiffness] Rayleigh Damping
  Earthquake Excitation with 1 Accelerogram Components
  Binary post-processor (.RES) file
  Small Displacement  analysis
```

STRUCTURAL DATA

```
  Number of Space Dimensions              2
  Number of Equations per Node            3
  Number of Nodes                         9
  No.Apparent Degrees of Freedom         27
  Number of Members                      12
  Number of Member Sections               6
  Number of Mode Shapes Required          5
  Pictures of Displaced Frame            NO
  Acceleration of Gravity           386.200
  % Critical Damping  Mode     1 =   5.00
  % Critical Damping  Mode     2 =   5.00
```

METHOD B
RUAUMOKO DATA ECHO
& MODAL RESULTS
– 4pages

TIME-HISTORY DATA

```
  Excitation Time-step            .01000 Seconds
  Duration of Excitation           2.000 Seconds
  Excitation Multiplier        1.000E+00
  No. Steps between Print-outs         5
  No. Steps between Disk Output        2
  No. Steps between Pictures           0
  ScreenPlot Mutiplier            10.000
  Max. X Displace.(Screen)         5.000
  Max. Y Displace.(Screen)         5.000
  Max. Cycles of Newton-Raphson        2
  Max. Cycles Damping Iteration        0
  Force Norm Limit             1.000E-03
  Wave Velocity - X axis.      0.000E+00
  Wave Velocity - Y axis.      0.000E+00
  1st Quake component from X        .000 Degrees
  DYNAPLOT Nodal Output: Displacement,Damping Force,Inertia Force & Load
```

1NODE	POSITION OF NODES		NODE FIXITY			MASTER NODE			OUTPUT
No.	X Co-ord	Y Co-ord	X	Y	Z	X	Y	Z	Flag
1	-1.3860E+03	5.7050E+02	0	0	0	0	0	0	0
2	-1.3960E+03	5.7050E+02	1	1	1	0	0	0	0
3	1.3860E+03	5.7050E+02	0	0	0	0	0	0	0
4	1.3760E+03	5.7050E+02	1	1	1	0	0	0	0
5	-1.3860E+03	-2.5130E+02	0	0	0	0	0	0	0
6	-1.3960E+03	-2.5130E+02	1	1	1	0	0	0	0
7	1.3860E+03	-2.5130E+02	0	0	0	0	0	0	0
8	1.3760E+03	-2.5130E+02	1	1	1	0	0	0	0

9 0.0000E+00 0.0000E+00 0 0 0 0 0 0 0

MEMBER Number	SECTION Number	NODE End1	NODE End2	NODE Inner1	NODE Inner2	NODE zz-axis	OUTPUT Flag
1	1	1	2	1	2		0
2	1	3	4	3	4		0
3	2	5	6	5	6		0
4	2	7	8	7	8		0
5	5	1	3	1	3		0
6	5	5	7	5	7		0
7	3	1	5	1	5		0
8	3	3	7	3	7		0
9	6	1	9	1	9		0
10	6	3	9	3	9		0
11	4	5	9	5	9		0
12	4	7	9	7	9		0

1 MEMBER PROPERTIES TABLE

=======================

0SECTION NUMBER 1 TYPE=SPRING

Spring Stiffness in X . = 2.800E+03
Rotational Stiffness. . = 1.000E+04
Bilinear Factor Transln.= .100
Angle Global-Local axes = .000

Spring Stiffness in Y . = 2.800E+03
Weight/(Unit Length). . = 0.000E+00
Bilinear Factor Rotation= 1.000

3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS

Positive X Force. . . . = 3.500E+02
Positive Y Force. . . . = 3.500E+02
Positive Moment . . . . = 3.500E+02

Negative X Force. . . . =-3.500E+02
Negative Y Force. . . . =-3.500E+02
Negative Moment . . . . =-3.500E+02

Modified Bi-linear TAKEDA Hystersis
ALPHA (Unloading) . . . = .5000
N Power Factor. . . . . = 1.

BETA (Reloading) . . . = .5000
Unload (1=DRAIN;2=EMORI)= 2.

0SECTION NUMBER 2 TYPE=SPRING

Spring Stiffness in X . = 8.150E+01
Rotational Stiffness. . = 1.000E+03
Bilinear Factor Transln.= .100
Angle Global-Local axes = .000

Spring Stiffness in Y . = 8.150E+01
Weight/(Unit Length). . = 0.000E+00
Bilinear Factor Rotation= 1.000

3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS

Positive X Force. . . . = 4.000E+01
Positive Y Force. . . . = 4.000E+01
Positive Moment . . . . = 4.000E+01

Negative X Force. . . . =-4.000E+01
Negative Y Force. . . . =-4.000E+01
Negative Moment . . . . =-4.000E+01

Modified Bi-linear TAKEDA Hystersis
ALPHA (Unloading) . . . = .5000
N Power Factor. . . . . = 1.

BETA (Reloading) . . . = .5000
Unload (1=DRAIN;2=EMORI)= 2.

0SECTION NUMBER 3 TYPE=FRAME -GIBERSON BEAM

Elastic Modulus . . . . = 4.770E+04
Cross Sectional Area. . = 1.000E+05
Moment of Inertia . . . = 5.000E+05
End-block Length End 1 = 4.000E+02
Joint Flexibility End 1 = 0.000E+00
Perfect Hinge at End 1 = NO

Shear Modulus . . . . . = 2.000E+04
Shear Area. . . . . . . = 0.000E+00
Weight/(Unit Length). . = 0.000E+00
End-block Length End 2 = 4.000E+02
Joint Flexibility End 2 = 0.000E+00
Perfect Hinge at End 2 = NO

```
   Linear Elastic Hysteresis

0SECTION NUMBER    4 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 7.000E+02    End-block Length  End 2 = 7.000E+02
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =        NO    Perfect Hinge  at End 2 =        NO

   Linear Elastic Hysteresis

0SECTION NUMBER    5 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 1.350E+03    End-block Length  End 2 = 1.350E+03
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =        NO    Perfect Hinge  at End 2 =        NO

   Linear Elastic Hysteresis

0SECTION NUMBER    6 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 7.450E+02    End-block Length  End 2 = 7.450E+02
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =        NO    Perfect Hinge  at End 2 =        NO

   Linear Elastic Hysteresis

1          LUMPED WEIGHT AT NODE
 Node       X-Weight   Y-Weight   Z-Rotatn
    1      3.061E+03  3.061E+03  3.061E+03
    3      3.061E+03  3.061E+03  3.061E+03
    5      7.163E+03  7.163E+03  7.163E+03
    7      7.163E+03  7.163E+03  7.163E+03
1          STATIC LOADING
 Node        X-Force    Y-Force   Z-Moment
1

 EXCITATION COMPONENT  1 is in "  BERG  " FORMAT
 In FILE. . . . . .EL40NSC.EQB
 First Line Number.=          1
 Digitizing DT. . .=       .010
 1/(Scale-Factor) .=  1.000E+00
 Excitation zero padded after End-of-File
 Initial Velocity  =       .000
 Initial Displacemt=       .000
 Time Scale        =      1.000
1FINAL TOTAL "LUMPED" NODAL WEIGHT
 ---------------------------------
  NODE      X Dirtn.   Y Dirtn.    Theta-Z
     1 3.0610E+03 3.0610E+03 3.0610E+03
     3 3.0610E+03 3.0610E+03 3.0610E+03
     5 7.1630E+03 7.1630E+03 7.1630E+03
```

7	7.1630E+03	7.1630E+03	7.1630E+03
Total	2.0448E+04	2.0448E+04	

1 NATURAL FREQUENCIES

MODE	Frequency	Period	% Damping	Damped Freq
1	1.346E+00	7.430E-01	5.000E+00	1.344E+00
2	1.660E+00	6.022E-01	5.000E+00	1.658E+00
3	1.987E+00	5.034E-01	5.175E+00	1.984E+00
4	6.353E+02	1.574E-03	1.057E+03	Complex
5	9.293E+02	1.076E-03	1.546E+03	Complex
6	9.318E+02	1.073E-03	1.550E+03	Complex
7	1.182E+03	8.457E-04	1.967E+03	Complex
8	1.204E+03	8.305E-04	2.003E+03	Complex
9	1.526E+05	6.553E-06	2.538E+05	Complex
10	2.453E+05	4.077E-06	4.080E+05	Complex
11	7.075E+05	1.413E-06	1.177E+06	Complex
12	7.514E+05	1.331E-06	1.250E+06	Complex

RAYLEIGH DAMPING Alpha= 4.67048E-01 Beta= 5.29414E-03

1 MODAL PROPERTIES - 1 Excitation Components

MODE	Part-Fact	Eff-Mass	%-M
1	6.421E-01	2.949E+01	56
2	2.008E-11	2.135E-20	56
3	6.421E-01	2.345E+01	100
4	-4.700E-11	1.324E-20	100
5	2.876E-06	3.696E-10	100

1 NORMAL MODES OF THE FRAME

NODE	D.o.f.	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
1	X-Disp.	5.573E-01	-8.821E-07	1.000E+00	-4.203E-01	7.925E-01
1	Y-Disp.	-7.466E-01	1.000E+00	7.466E-01	1.826E-02	1.000E+00
1	Z-Rotn.	5.386E-04	4.160E-09	-5.386E-04	1.402E-03	-2.308E-03
3	X-Disp.	5.573E-01	8.822E-07	1.000E+00	4.203E-01	7.925E-01
3	Y-Disp.	7.466E-01	1.000E+00	-7.466E-01	1.826E-02	-1.000E+00
3	Z-Rotn.	5.386E-04	-4.160E-09	-5.386E-04	-1.402E-03	-2.308E-03
5	X-Disp.	1.000E+00	-8.930E-07	5.574E-01	2.930E-01	-3.387E-01
5	Y-Disp.	-7.466E-01	1.000E+00	7.466E-01	-7.802E-03	-6.281E-01
5	Z-Rotn.	5.386E-04	-8.144E-09	-5.386E-04	1.459E-03	1.532E-04
7	X-Disp.	1.000E+00	8.930E-07	5.574E-01	-2.930E-01	-3.387E-01
7	Y-Disp.	7.466E-01	1.000E+00	-7.466E-01	-7.802E-03	6.281E-01
7	Z-Rotn.	5.386E-04	8.144E-09	-5.386E-04	-1.459E-03	1.532E-04
9	X-Disp.	8.646E-01	2.007E-11	6.927E-01	-6.313E-12	-6.555E-02
9	Y-Disp.	-2.018E-11	1.000E+00	-1.110E-11	1.000E+00	-2.587E-10
9	Z-Rotn.	5.386E-04	2.478E-15	-5.386E-04	-3.406E-15	6.466E-04

```
METHOD D ANALYSIS OF NAVY WHARF (Inelastic Time History)
2 0 1 0 0 0
20 29 9 6 1 3 386.2 2. 2. 0.01 20 1.5
5 2 0 1 1. 10 5. 5.
2
NODES 0
1 -3786  570.5 0 0 0
2 -3796  570.5 1 1 1
3 -1014  570.5 0 0 0
4 -1024  570.5 1 1 1
5 -3786 -251.3 0 0 0
6 -3796 -251.3 1 1 1
7 -1014 -251.3 0 0 0
8 -1024 -251.3 1 1 1
9 -2400. 0. 0 0 0 0 0 0 3
10 -5 0 0 0 0 0 0 0 3
11 1014 570.5 0 0 0
12 1004 570.5 1 1 1
13 3786 570.5 0 0 0
14 3776 570.5 1 1 1
15 1014 -251.3 0 0 0
16 1004 -251.3 1 1 1
17 3786 -251.3 0 0 0
18 3776 -251.3 1 1 1
19 2400. 0. 0 0 0 0 0 0 3
20 5 0. 0 0 0 0 0 0 3
ELEMENTS 3
1 1 1 2  0 0 0
2 1 3 4
3 2 5 6
4 2 7 8
5 5 1 3
6 5 5 7
7 3 1 5
8 3 3 7
9 6 1 9
10 6 3 9
11 4 5 9
12 4 7 9
13 1 11 12
14 1 13 14 0 0 0
15 2 15 16
16 2 17 18
17 5 11 13
18 5 15 17
19 3 11 15
20 3 13 17
21 6 11 19
22 6 13 19
23 4 15 19
24 4 17 19
25 7 3 10
26 7 11 20
27 8 7 10
28 8 15 20
29 9 10 20  0 0 0
PROPS
1 SPRING
1 4 0 0 2800. 2800. 10000. 0. 0.0553 1.
3630 -3630 3630 -3630 3630000 -3630000
```

$\boxed{\text{METHOD D INPUT DATA}}$

2 pages.

```
0.5 0.5 1 2
2 SPRING
1 4 0 0 81.5 81.5 1000. 0. 0.631 1.
326 -326 326 -326 326000 -326000
0.5 0.5 1 2
3 FRAME
1 0 0 0
47700. 20000. 100000. 0. 500000. 0. 400. 400.
4 FRAME
1
47700. 20000. 100000. 0. 500000. 0. 700. 700.
5 FRAME
1
47700.  20000. 100000. 0. 500000. 0. 1350 1350
6 FRAME
1
47700. 20000. 100000. 0. 500000. 0. 745. 745.
7 FRAME
1
47700. 20000. 100000. 0. 500000. 0. 575. 575.
8 FRAME
1
47700. 20000. 100000. 0. 500000. 0. 515. 515.
9 FRAME
1 1
47700. 20000. 100000. 0. 500000.
WEIGHTS 0
1 3061 3061 3061
3 3061 3061 3061
5 7163 7163 7163
7 7163 7163 7163
11 3061 3061 3061
13 3061 3061 3061
15 7163 7163 7163
17 7163 7163 7163
LOADS
1 0. 0. 0.
EQUAKE
0 1 0.01 1.0 -1
START
```

DATE: 16 FEBRUARY 2000 TIME 8:28: .46

STRUCTURE

METHOD D ANALYSIS OF NAVY WHARF (Inelastic Time History)

ANALYSIS DETAILS

Analysis Type : In-elastic
Time Variation: Time-history - Newmark (Beta = 0.25)
Mass Matrix : Lumped
Damping Matrix: Alpha*[Mass]+Beta*[Initial Stiffness] Rayleigh Damping
Earthquake Excitation with 1 Accelerogram Components
Binary post-processor (.RES) file
Small Displacement analysis

METHOD D
RUAUMOKO DATA ECHO
+MODAL RESULTS

6 pages

STRUCTURAL DATA

Number of Space Dimensions	2
Number of Equations per Node	3
Number of Nodes	20
No.Apparent Degrees of Freedom	60
Number of Members	29
Number of Member Sections	9
Number of Mode Shapes Required	6
Pictures of Displaced Frame	NO
Acceleration of Gravity	386.200
% Critical Damping Mode 1 =	2.00
% Critical Damping Mode 3 =	2.00

TIME-HISTORY DATA

Excitation Time-step	.01000 Seconds
Duration of Excitation	20.000 Seconds
Excitation Multiplier	1.500E+00
No. Steps between Print-outs	5
No. Steps between Disk Output	2
No. Steps between Pictures	0
ScreenPlot Mutiplier	10.000
Max. X Displace.(Screen)	5.000
Max. Y Displace.(Screen)	5.000
Max. Cycles of Newton-Raphson	2
Max. Cycles Damping Iteration	0
Force Norm Limit	1.000E-03
Wave Velocity - X axis.	0.000E+00
Wave Velocity - Y axis.	0.000E+00
1st Quake component from X	.000 Degrees

DYNAPLOT Nodal Output: Displacement,Damping Force,Inertia Force & Load

1NODE	POSITION OF NODES		NODE FIXITY			MASTER NODE			OUTPUT
No.	X Co-ord	Y Co-ord	X	Y	Z	X	Y	Z	Flag
1	-3.7860E+03	5.7050E+02	0	0	0	0	0	0	0
2	-3.7960E+03	5.7050E+02	1	1	1	0	0	0	0
3	-1.0140E+03	5.7050E+02	0	0	0	0	0	0	0
4	-1.0240E+03	5.7050E+02	1	1	1	0	0	0	0
5	-3.7860E+03	-2.5130E+02	0	0	0	0	0	0	0
6	-3.7960E+03	-2.5130E+02	1	1	1	0	0	0	0
7	-1.0140E+03	-2.5130E+02	0	0	0	0	0	0	0
8	-1.0240E+03	-2.5130E+02	1	1	1	0	0	0	0

9	-2.4000E+03	0.0000E+00	0	0	0	0	0	0	3
10	-5.0000E+00	0.0000E+00	0	0	0	0	0	0	3
11	1.0140E+03	5.7050E+02	0	0	0	0	0	0	0
12	1.0040E+03	5.7050E+02	1	1	1	0	0	0	0
13	3.7860E+03	5.7050E+02	0	0	0	0	0	0	0
14	3.7760E+03	5.7050E+02	1	1	1	0	0	0	0
15	1.0140E+03	-2.5130E+02	0	0	0	0	0	0	0
16	1.0040E+03	-2.5130E+02	1	1	1	0	0	0	0
17	3.7860E+03	-2.5130E+02	0	0	0	0	0	0	0
18	3.7760E+03	-2.5130E+02	1	1	1	0	0	0	0
19	2.4000E+03	0.0000E+00	0	0	0	0	0	0	3
20	5.0000E+00	0.0000E+00	0	0	0	0	0	0	3

1

MEMBER Number	SECTION Number	NODE End1	NODE End2	NODE Inner1	NODE Inner2	NODE zz-axis	OUTPUT Flag
1	1	1	2	1	2		0
2	1	3	4	3	4		3
3	2	5	6	5	6		3
4	2	7	8	7	8		3
5	5	1	3	1	3		3
6	5	5	7	5	7		3
7	3	1	5	1	5		3
8	3	3	7	3	7		3
9	6	1	9	1	9		3
10	6	3	9	3	9		3
11	4	5	9	5	9		3
12	4	7	9	7	9		3
13	1	11	12	11	12		3
14	1	13	14	13	14		0
15	2	15	16	15	16		3
16	2	17	18	17	18		3
17	5	11	13	11	13		3
18	5	15	17	15	17		3
19	3	11	15	11	15		3
20	3	13	17	13	17		3
21	6	11	19	11	19		3
22	6	13	19	13	19		3
23	4	15	19	15	19		3
24	4	17	19	17	19		3
25	7	3	10	3	10		3
26	7	11	20	11	20		3
27	8	7	10	7	10		3
28	8	15	20	15	20		3
29	9	10	20	10	20		0

1

MEMBER PROPERTIES TABLE

====================

0SECTION NUMBER 1 TYPE=SPRING

Spring Stiffness in X . = 2.800E+03
Rotational Stiffness. . = 1.000E+04
Bilinear Factor Transln.= .055
Angle Global-Local axes = .000

Spring Stiffness in Y . = 2.800E+03
Weight/(Unit Length). . = 0.000E+00
Bilinear Factor Rotation= 1.000

3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS

Positive X Force. . . . = 3.630E+03
Positive Y Force. . . . = 3.630E+03
Positive Moment . . . . = 3.630E+06

Negative X Force. . . . =-3.630E+03
Negative Y Force. . . . =-3.630E+03
Negative Moment . . . . =-3.630E+06

Modified Bi-linear TAKEDA Hysteresis
ALPHA (Unloading) . . . = .5000

BETA (Reloading) . . . = .5000

```
   N Power Factor. . . . . =          1.    Unload (1=DRAIN;2=EMORI)=         2.

0SECTION NUMBER    2 TYPE=SPRING
 ------------------------------
   Spring Stiffness in X . = 8.150E+01    Spring Stiffness in Y . = 8.150E+01
   Rotational Stiffness. . = 1.000E+03    Weight/(Unit Length). . = 0.000E+00
   Bilinear Factor Transln.=       .631    Bilinear Factor Rotation=     1.000
   Angle Global-Local axes =       .000

   3 SPRING COMPONENTS - NO INTERACTION - YIELD FORCES AND MOMENTS
   Positive X Force. . . . = 3.260E+02    Negative X Force. . . . =-3.260E+02
   Positive Y Force. . . . = 3.260E+02    Negative Y Force. . . . =-3.260E+02
   Positive Moment . . . . = 3.260E+05    Negative Moment . . . . =-3.260E+05

   Modified Bi-linear TAKEDA Hystersis
   ALPHA (Unloading) . . . =      .5000    BETA  (Reloading) . . . =      .5000
   N Power Factor. . . . . =          1.    Unload (1=DRAIN;2=EMORI)=         2.

0SECTION NUMBER    3 TYPE=FRAME -GIBERSON BEAM
 ---------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 4.000E+02    End-block Length  End 2 = 4.000E+02
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =         NO    Perfect Hinge  at End 2 =         NO

   Linear Elastic Hysteresis

0SECTION NUMBER    4 TYPE=FRAME -GIBERSON BEAM
 ---------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 7.000E+02    End-block Length  End 2 = 7.000E+02
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =         NO    Perfect Hinge  at End 2 =         NO

   Linear Elastic Hysteresis

0SECTION NUMBER    5 TYPE=FRAME -GIBERSON BEAM
 ---------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 1.350E+03    End-block Length  End 2 = 1.350E+03
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =         NO    Perfect Hinge  at End 2 =         NO

   Linear Elastic Hysteresis

0SECTION NUMBER    6 TYPE=FRAME -GIBERSON BEAM
 ---------------------------
   Elastic Modulus . . . . = 4.770E+04    Shear Modulus . . . . . = 2.000E+04
   Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . = 0.000E+00
   Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
   End-block Length  End 1 = 7.450E+02    End-block Length  End 2 = 7.450E+02
   Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
   Perfect Hinge  at End 1 =         NO    Perfect Hinge  at End 2 =         NO
```

```
    Linear Elastic Hysteresis

0SECTION NUMBER    7 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
    Elastic Modulus . . . . . = 4.770E+04    Shear Modulus . . . . . . = 2.000E+04
    Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . . = 0.000E+00
    Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
    End-block Length  End 1 = 5.750E+02    End-block Length  End 2 = 5.750E+02
    Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
    Perfect Hinge  at End 1 =        NO    Perfect Hinge  at End 2 =        NO

    Linear Elastic Hysteresis

0SECTION NUMBER    8 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
    Elastic Modulus . . . . . = 4.770E+04    Shear Modulus . . . . . . = 2.000E+04
    Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . . = 0.000E+00
    Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
    End-block Length  End 1 = 5.150E+02    End-block Length  End 2 = 5.150E+02
    Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
    Perfect Hinge  at End 1 =        NO    Perfect Hinge  at End 2 =        NO

    Linear Elastic Hysteresis

0SECTION NUMBER    9 TYPE=FRAME -GIBERSON BEAM
 ----------------------------
    Elastic Modulus . . . . . = 4.770E+04    Shear Modulus . . . . . . = 2.000E+04
    Cross Sectional Area. . = 1.000E+05    Shear Area. . . . . . . . = 0.000E+00
    Moment of Inertia . . . = 5.000E+05    Weight/(Unit Length). . = 0.000E+00
    End-block Length  End 1 = 0.000E+00    End-block Length  End 2 = 0.000E+00
    Joint Flexibility End 1 = 0.000E+00    Joint Flexibility End 2 = 0.000E+00
    Perfect Hinge  at End 1 =       YES    Perfect Hinge  at End 2 =        NO

    Linear Elastic Hysteresis

1          LUMPED WEIGHT AT NODE
 Node        X-Weight    Y-Weight    Z-Rotatn
    1      3.061E+03   3.061E+03   3.061E+03
    3      3.061E+03   3.061E+03   3.061E+03
    5      7.163E+03   7.163E+03   7.163E+03
    7      7.163E+03   7.163E+03   7.163E+03
   11      3.061E+03   3.061E+03   3.061E+03
   13      3.061E+03   3.061E+03   3.061E+03
   15      7.163E+03   7.163E+03   7.163E+03
   17      7.163E+03   7.163E+03   7.163E+03
1          STATIC LOADING
 Node         X-Force     Y-Force    Z-Moment
1

 EXCITATION COMPONENT  1 is in "  BERG  " FORMAT
 In FILE. . . . . .EL40NSC.EQB
 First Line Number.=           1
 Digitizing DT. . .=        .010
 1/(Scale-Factor) .=   1.000E+00
 Excitation zero padded after End-of-File
 Initial Velocity  =        .000
 Initial Displacemt=        .000
 Time Scale        =       1.000
1FINAL TOTAL "LUMPED" NODAL WEIGHT
 ----------------------------------
```

NODE	X Dirtn.	Y Dirtn.	Theta-Z
1	3.0610E+03	3.0610E+03	3.0610E+03
3	3.0610E+03	3.0610E+03	3.0610E+03
5	7.1630E+03	7.1630E+03	7.1630E+03
7	7.1630E+03	7.1630E+03	7.1630E+03
11	3.0610E+03	3.0610E+03	3.0610E+03
13	3.0610E+03	3.0610E+03	3.0610E+03
15	7.1630E+03	7.1630E+03	7.1630E+03
17	7.1630E+03	7.1630E+03	7.1630E+03
Total	4.0896E+04	4.0896E+04	

1 NATURAL FREQUENCIES

MODE	Frequency	Period	% Damping	Damped Freq
1	1.498E+00	6.676E-01	2.000E+00	1.498E+00
2	1.660E+00	6.022E-01	1.996E+00	1.660E+00
3	1.730E+00	5.780E-01	2.000E+00	1.730E+00
4	1.826E+00	5.476E-01	2.011E+00	1.826E+00
5	2.615E+02	3.823E-03	1.621E+02	Complex
6	4.123E+02	2.425E-03	2.555E+02	Complex
7	6.456E+02	1.549E-03	4.000E+02	Complex
8	7.802E+02	1.282E-03	4.834E+02	Complex
9	9.390E+02	1.065E-03	5.818E+02	Complex
10	9.473E+02	1.056E-03	5.869E+02	Complex
11	1.143E+03	8.751E-04	7.081E+02	Complex
12	1.151E+03	8.686E-04	7.134E+02	Complex
13	1.234E+03	8.103E-04	7.647E+02	Complex
14	1.257E+03	7.953E-04	7.791E+02	Complex
15	1.649E+03	6.063E-04	1.022E+03	Complex
16	1.653E+03	6.049E-04	1.024E+03	Complex
17	1.527E+05	6.547E-06	9.464E+04	Complex
18	1.527E+05	6.547E-06	9.464E+04	Complex
19	2.472E+05	4.046E-06	1.532E+05	Complex
20	2.472E+05	4.045E-06	1.532E+05	Complex
21	7.265E+05	1.376E-06	4.501E+05	Complex
22	7.266E+05	1.376E-06	4.502E+05	Complex
23	8.608E+05	1.162E-06	5.334E+05	Complex
24	8.643E+05	1.157E-06	5.355E+05	Complex

RAYLEIGH DAMPING Alpha= 2.01762E-01 Beta= 1.97228E-03

1 MODAL PROPERTIES - 1 Excitation Components

MODE	Part-Fact	Eff-Mass	%-M
1	-6.766E-01	5.606E+01	53
2	-2.367E-11	5.930E-20	53
3	2.502E-03	2.373E-04	53
4	6.781E-01	4.983E+01	100
5	1.161E-05	8.151E-09	100
6	-1.176E-09	9.629E-17	100

1 NORMAL MODES OF THE FRAME

NODE	D.o.f.	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
1	X-Disp.	-6.300E-01	8.174E-06	-2.367E-01	8.470E-01	4.403E-01	8.152E-01
1	Y-Disp.	1.000E+00	1.000E+00	-5.749E-01	1.000E+00	2.176E-01	6.661E-02
1	Z-Rotn.	-2.648E-04	-1.139E-08	4.165E-04	-2.657E-04	-1.174E-03	1.380E-04
3	X-Disp.	-6.300E-01	9.211E-06	-2.367E-01	8.470E-01	-9.499E-02	7.818E-01
3	Y-Disp.	2.660E-01	9.999E-01	5.797E-01	2.633E-01	-9.660E-01	1.894E-02
3	Z-Rotn.	-2.648E-04	-1.594E-08	4.165E-04	-2.658E-04	5.461E-04	-2.930E-04
5	X-Disp.	-8.476E-01	-5.233E-06	1.056E-01	6.286E-01	-2.337E-01	1.000E+00
5	Y-Disp.	1.000E+00	1.000E+00	-5.749E-01	1.000E+00	2.515E-01	-2.689E-02

5	Z-Rotn.	-2.648E-04	-2.381E-08	4.165E-04	-2.658E-04	-1.242E-03	3.617E-04
7	X-Disp.	-8.476E-01	-4.187E-06	1.056E-01	6.286E-01	8.954E-02	5.729E-01
7	Y-Disp.	2.660E-01	1.000E+00	5.797E-01	2.633E-01	-9.973E-01	-8.087E-03
7	Z-Rotn.	-2.648E-04	-1.458E-08	4.165E-04	-2.658E-04	5.432E-04	-2.566E-04
9	X-Disp.	-7.810E-01	-1.544E-07	9.001E-04	6.954E-01	-1.308E-02	7.799E-01
9	Y-Disp.	6.330E-01	1.000E+00	2.432E-03	6.316E-01	-9.832E-01	1.942E-01
9	Z-Rotn.	-2.648E-04	-1.717E-08	4.165E-04	-2.658E-04	-5.239E-04	1.361E-06
10	X-Disp.	-7.810E-01	-7.812E-11	8.980E-04	6.954E-01	8.486E-02	2.940E-01
10	Y-Disp.	-1.210E-03	9.999E-01	1.000E+00	-4.886E-03	-3.788E-01	-5.293E-01
10	Z-Rotn.	-2.648E-04	-1.712E-08	4.165E-04	-2.658E-04	5.374E-04	-8.044E-04
11	X-Disp.	-6.312E-01	-9.211E-06	2.387E-01	8.440E-01	-9.790E-02	-7.791E-01
11	Y-Disp.	-2.689E-01	9.999E-01	5.753E-01	-2.705E-01	9.686E-01	1.611E-02
11	Z-Rotn.	-2.627E-04	1.594E-08	-4.168E-04	-2.606E-04	5.336E-04	2.843E-04
13	X-Disp.	-6.312E-01	-8.174E-06	2.387E-01	8.440E-01	4.385E-01	-8.181E-01
13	Y-Disp.	-9.971E-01	1.000E+00	-5.801E-01	-9.928E-01	-2.201E-01	6.716E-02
13	Z-Rotn.	-2.627E-04	1.139E-08	-4.168E-04	-2.606E-04	-1.184E-03	-1.286E-04
15	X-Disp.	-8.471E-01	4.187E-06	-1.038E-01	6.299E-01	8.817E-02	-5.746E-01
15	Y-Disp.	-2.689E-01	1.000E+00	5.753E-01	-2.705E-01	1.000E+00	-1.116E-02
15	Z-Rotn.	-2.627E-04	1.458E-08	-4.168E-04	-2.606E-04	5.309E-04	2.479E-04
17	X-Disp.	-8.471E-01	5.233E-06	-1.038E-01	6.299E-01	-2.371E-01	-9.982E-01
17	Y-Disp.	-9.971E-01	1.000E+00	-5.801E-01	-9.928E-01	-2.542E-01	-2.600E-02
17	Z-Rotn.	-2.627E-04	2.381E-08	-4.168E-04	-2.606E-04	-1.253E-03	-3.517E-04
19	X-Disp.	-7.810E-01	1.543E-07	8.931E-04	6.954E-01	-1.570E-02	-7.798E-01
19	Y-Disp.	-6.330E-01	1.000E+00	-2.400E-03	-6.316E-01	9.826E-01	1.867E-01
19	Z-Rotn.	-2.627E-04	1.717E-08	-4.168E-04	-2.606E-04	-5.161E-04	6.249E-07
20	X-Disp.	-7.810E-01	3.074E-11	8.952E-04	6.954E-01	8.451E-02	-2.947E-01
20	Y-Disp.	-3.827E-03	9.999E-01	9.958E-01	-7.503E-03	3.729E-01	-5.240E-01
20	Z-Rotn.	-2.627E-04	1.712E-08	-4.168E-04	-2.606E-04	5.677E-04	7.977E-04